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BACKFILL ALTERNATIVES IN ONTARIO MINES

VOLUME 1 - REPORT

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EXECUTIVE SUMMARY

INTRODUCTION

This report presents the results of a study on "Backfill Alternatives in Ontario Mines". The project was completed for the Department of Supply and Services Canada (DSS) through the Canada Centre for Mineral & Energy Technology (CANMET) under contract serial number DSS No. 09SQ. 23440-6-9011. The project was conducted under the Canada/Ontario Mineral Development Agreement (C.O.M.D.A.)

The mining of rich, massive ore bodies in Canada today poses problems of scale and sequencing. Huge checkerboard patterns of stopes and pillars are developed over a time frame of a decade or more for any given level. This calls for considerable foresight in backfill support design. Backfilling can represent a significant operating expense to mining operations, especially when conditions require the use of high strength fills. Thus there is a considerable incentive for the development of low cost alternative backfills and cements.

The objectives of this project were;

1. To determine the relative merits of various backfilling alternative, including high modulus fill, to underground stability in mining a massive ore deposit.

2. To establish general engineering specifications for a range of high quality, low placement cost backfills applicable to the bulk mining of rich ore bodies.

As background to the research work a review of backfill engineering principles is presented along with some simple design methods.

A series of site visits was conducted to various mines in Ontario and Quebec to review the current state of backfilling operations in Canadian mines.

A short review of both conventional hydraulic backfill and rockfill theory is presented. Some practical aspects of transport, placement, and drainage are discussed. Typical engineering properties of these backfills are presented.

A literature review of several innovative and less common backfilling techniques was conducted. This included an investigation of the paste and dense hydraulic backfills which are state of the art in tailings backfilling. Frozen and pneumatic backfilling systems were also reviewed.

As part of a subcontract to the project, a laboratory centrifuge at Queens University was used to develop design specifications for several innovative backfilling techniques which have some potential to reduce fill costs. These techniques included reinforcement of backfill with geotextiles and the placement of layered and sloped wall fill. The scale model work indicated that significant improvements in fill stability could be achieved with such techniques.

The potential of monolithic packing systems which are commonly used in coal and soft rock mining was investigated. An extensive series of laboratory strength tests were conducted as a subcontract to the project on various monolithic packing materials. A range of engineering specifications were determined and optimum mixtures identified. An underground trial using monolithic packing materials to build a fill fence was conducted. Similar materials were also used as capping in a rockfilled stope to aid in getting fill tight to the stope backs.

The instrumentation of backfill and rock in situ is important to the understanding of how the fill behaves in the mining environment. The in situ monitoring and instrumentation of mine backfill conducted for the project is included as a series of case studies. Also included are selected case studies of previous backfill instrumentation by Falconbridge and Kidd Creek Mines.

An extensive amount of research laboratory test work was conducted on the performance of binder alternatives for both hydraulic and rockfill. The results presented include work with ground ferrous blast furnace slag, fly ash, ground copper refinery slag and anhydrite. Various methods of activating these pozzolanic and cementitious materials were investigated.

PROJECT CONCLUSIONS AND RECOMMENDATIONS

The Backfill Alternatives in Ontario Mines, encompased 5 years of work which includes an extensive literature review, a survey of backfilling operations at several mines in Ontario and Quebec, a series of extensive laboratory tests which investigated various backfill alternatives, field trials of these alternatives, and experience with instrumentation of backfill. This work has indicated several ways in which backfill operations can be improved, as well as indicating the directions in which future research might give the best returns.

Binders account for much of the cost in placing backfill. Hence much of the emphasis in this project was placed upon the search for and analysis of binder alternatives. A significant amount of laboratory testing of chemical additives was conducted while investigating ways to get optimum performance from these alternatives. The following binders were considered in the project for use with hydraulic tailings or rockfill.

1. Ordinary portland cement. The industry standard binder against which others were compared.

2. Ground ferrous blast furnace slag. Several slags from different sources were tested as a binder for both hydraulic fill and rockfill. The results of laboratory test work indicated that ferrous slag could be used as a backfill binder, giving ultimate compressive strengths of backfill equal to or greater than those achieved with portland cement. A significant difference in the performance of different slags was noticed. The major problem with the use of ferrous slag is its slow rate of strength development. As a consequence of the test work, Falconbridge Limited have use ferrous slag extensively as a binder for hydraulic tailings backfill at their Sudbury Operations. Ferrous slag is also used extensively in other Ontario mines. Slag is available at a significant price discount over portland cement.

3. Flyash, a waste product from coal fired power stations, was tested and found to be a suitable binder for both hydraulic and rockfill. Two types of ash were investigated, type C which gave superior performance and the less successful type F. Type C flyash is used in a blend with portland cement to make an overall low cost binder, up to 60% flyash has been used successfully. The strength of backfill consolidated with a flyash blend binder can exceed that achieved with portland cement at lower cost but again the rate of strength development is

slower. Kidd Creek Mines and several other mines in Ontario and Quebec use flyash blend binders for consolidating rockfill.

4. Ground Kidd Creek copper slag was investigated for its potential as a backfill binder. The test work indicated that it could be used successfully when blended with portland cement, up to about 40% slag could be used. Costs involved in grinding of the slag to the size required to achieve sufficient reactivity have not encouraged its use, especially when it is competing in cost against ferrous slag and flyash. Despite this, description of the test work is useful as a guide to other operations embarking on investigations into the potential of their slag.

5. Anhydrite was tested in the laboratory for use as a hydraulic tailings fill binder. It proved to be a successful binder capable of producing a high strength backfill, but the test work used binder addition rates which would be practically unrealistic. This research work is of academic interest only at this stage, more work using realistically economic addition rates is required.

Additions of anhydrite, gypsum, and lime to blended binders of copper slag with portland cement and flyash with portland cement resulted in increased compressive strength of backfill. The increases were often significant, the best result was obtained when 3% anhydrite was added to a slag mix which caused strengths increase of 217% and 240% after 28 and 56 days of curing respectively. The ability to improve the performance of copper slag and flyash by adding small amounts of anhydrite or gypsum is a significant conclusion of the project.

Various chemical accelerators commonly used in the concrete industry were tested with copper slag and portland cement blended binders.

1. Daracell, a product of W.R.Grace Chemical Company, performed well with the portland cement and copper slag binder. Pozzolith 133-HE, a product of the Master Builder Chemical Company, also performed well with the portland cement and copper slag binder. However the high cost of Daracell and Pozzolith makes the use of these chemicals impractical.

2. K_2SO_4 showed good potential. A 5% K_2SO_4 addition increased the strength of portland cement and copper slag mix by 40% after 56 days of curing.

3. The common accelerator used in concrete, $CaCl_2$, showed good potential. Although the improvement in strength this chemical was not as dramatic as others tested, its moderate cost make it a potentially useful additive. Test work indicated that 2% $CaCl_2$ increased the early

strength of a copper slag portland cement mix by 20%. This indicated that $CaCl_2$ could be used if a faster mining cycle is required.

The conclusion from the above results with accelerators and copper slag is significant. Mines employing the more commonly used ferrous slags as backfill binder could take the implications of these accelerator results as a starting point for investigations into improving the performance of their own product.

It has been recognized that there is potential for optimizing the performance of consolidated backfill in pillar recovery situations where a free standing wall of fill is exposed by using some innovative design approaches. A centrifuge modelling technique was used to investigate three of these innovations with consolidated hydraulic tailings fill.

1. Sloped wall fill. By excavating a stope with sloped walls, backfill may be placed so that at least part of its free standing wall is laid back from the vertical. Model studies of this design indicate that such a fill wall is more than twice as stable as theory predicts and more than three times as stable as theory predicts for a vertical fill wall.

2. Dense fill wall. When access is available it would be advantageous to place a wall of high strength fill against the pillar to be mined. When exposed, this high strength wall would be more stable than if the same total binder had been used to place a uniformly cemented fill. Model studies indicate that a dense fill wall is capable of standing and supporting unconsolidated fill behind it, which could result in significant binder savings .

3. Layered fill. This design concept has evolved from experience with cut and fill stoping where strong layers of highly cemented fill are laid as working floors over layers of weakly cemented bulk fill. Pillar extraction along side such stopes has shown that the wall exposures of such fill to be remarkably stable. Model studies indicate that layered fill systems are greatly superior to bulk pour systems. The potential for increasing the stable height of a fill exposure without increasing binder consumption is significant.

Centrifuge modelling techniques were also used to investigate the potential for the use of geotextile grids and elements to mechanically reinforce hydraulic tailings fill. These tests concluded that mesh elements could reinforce fill but the additions rates required are so high as to make their use uneconomic. Geotextile grids were excellent reinforcement and the practical problems of their placement should be investigated.

A significant conclusion of this report is that centrifugal modelling is a very useful method for low cost physical modelling of backfill designs.

Monolithic packing and other cementitious materials were reviewed, tested as part of this project. This is a technology transfer from the soft rock mining industry which successfully uses large quantities of these materials. It is concluded that these materials have limited specialized applications in the hard rock mining industry at the present time. The small quantities required make these materials very expensive in the applications tried here. Practical problems encountered with the use of small batches and lack of operator experience could be overcome should a application be found which could take advantage of economies of scale.

An extensive program of stope monitoring and instrumentation was carried out as part of the project. As expected, results indicated that a stope wall expanded adjacent to a mined-out area and contracted in the solid ground away from the opening. After each blast expansion continued throughout the wall opposite to the blasted area. Most of the recorded expansion took place after major blasts. After consolidated rockfill was placed, the expansion of the wall halted and inward movement began for up to 0.9 mm in the monitored stope. This implies that the placed rockfill developed an active pressure state compressing the relaxed stope wall.

Blast monitoring of consolidated rockfill at Kidd Creek Mines resulted in the following conclusions. Transmission coefficient from rock through consolidated rockfill averaged 37%. Absorption coefficient in consolidated rockfill was estimated at 0.41 db/m. P-wave velocity in consolidated rockfill averaged 3,220 m/s. An estimated particle velocity of 30 cm/s caused scabbing failure of a competent consolidated rock fill.

Testing indicated that the bearing plate-load test would provide a quick and reliable method for determining the in-situ strength of consolidated rockfill.

A review of alternative hydraulic fill placement technology was conducted. Known as paste fill and high density hydraulic fill these systems are designed to minimize the quantity of transport water required. This allows the fill to be designed to meet the strength and performance criteria required for optimum in situ performance, and not be compromised by transport and drainage requirements. The technology of low water content fill is reasonably well understood with a limited number of operations around the world employing these techniques. It is the dissemination of knowledge and the subsequent recognition of potential applications which is required for industry to gain from these new backfill alternatives.

RECOMMENDATIONS

The following recommendations have been suggested by the work carried out during the course of this project:

1. The instrumentation of backfill materials remains difficult and often expensive. An effort should be made to further develop the reliability and cost effectiveness of sampling and instrumentation in backfill. This will encourage more mines to conduct regular sampling and instrumentation which will, over time, build up a data base of fill behavior and in situ properties for better designs in the future.

2. A regular review of actual backfill operations in Canadian mines should be conducted. A report of the review should be disseminated throughout the industry. This would provide a valuable reference for design and improvements in backfilling technology. A less frequent review of selected overseas backfilling operations should be conducted to indentify trends and the development of new technologies.

3. Research into the effects of segregation on the mass strength of both hydraulic and rockfill is recommended. Segregation is commonly observed in all fill types and may play a more significant role in controlling overall mass strength of exposed fill than binder content in some circumstances. Optimized filling operations requires a better understanding of segregation effects, both positive and negative.

4. Investigations into the potential of high density hydraulic fill as a lower technology alternative to paste fill systems is recommended. A combination of high density hydraulic fill with the sloped wall or layered fill designs modelled in this project along with a low cost higher strength binder may give fill strengths equal or greater than those possible with paste fill, without the major technology changes required of a paste.

5. Rockfill is becoming a more popular fill type as strength and stiffness requirements increase with changes in mining methods and operating depths. It is recommended that more emphasis be placed on research into rockfill technology. This should include investigations into the optimum sizing of particles for rockfill.

6. The behavior of confined backfill placed as a stiff fill in order to limit stope convergence for control of rock bursting problems should be investigated. This should consider the role of both binders and particle sizing on confined stiffness. Much research has been done on determining the engineering properties of fill required to stand as a free face during pillar recovery

operations, but with the increasing requirements for placement of stiff fill there is a need to better understand the engineering characteristics of confined fill.

7. Ferrous slag and fly ash are now well established as cost effective alternatives to normal portland cement as binders in both hydraulic and rock fills in Ontario mines. At this point there seems to be little incentive to pursue investigations into additional alternative binders such as non ferrous slags. The selection of a backfill binder has become a site specific engineering and commercial problem rather than one requiring any fundamental research.

8. Further investigations into the potential of Tekfill type products is recommended. These products are routinely used in South African mining operations in conjunction with dense hydraulic fills. Development of denser hydraulic fills in Ontario mines may benefit from the use of similar products.

9. The application of monolithic packing technology to underground hard rock mining should be further investigated. The results of this project show that this technology, commonly used in coal and soft rock mining, has significant potential for specialized applications in Ontario mines.

10. The project has shown that there is significant potential for the use of chemical activators to modify the reactivity and curing rates of backfill binders. Further investigations into this potential is recommended in order to better determine the economics of using various chemicals as activators.

11. Many of the instruments used for determination of the inplace characteristics of backfill were developed for use in rock which is much less prone to disturbance. Work should be conducted on proper instrumentation placement procedures for instruments to ensure instrument reliability.

DISCLAIMER

The results presented in this report are considered particular to the materials being tested. Falconbridge Limited does not endorse any of the products mentioned in this report.

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RESUME

INTRODUCTION

Le présent rapport expose les résultats d'une étude ayant pour objet les "Liants alternatifs pour consolidation des remblais dans les mines de l'Ontario", projet réalisé pour le département d'Aprovisionnements et services (DSS) par l'entremise du Centre canadien de la technologie des minéraux et de l'énergie (CANMET) aux termes du contrat no. 09SQ-23440-6-9011. Ce projet s'est réalisé dans le cadre du C.O.M.D.A. (Entente Canada/Ontario sur le développement minéral).

Au Canada, l'exploitation de gîtes minéralisés massifs comporte les problèmes d'ordre de dimensionnement et d'ordonnancement. En une dizaine d'annees, des réseaux énormes de chambres et de piliers se sont développés à chaque niveau de profondeur, ce qui exige que l'on prenne beaucoup de précautions dans la conception des remblais solides. Le remblayage peut représenter une augmentation considérable du coût des opérations minières, surtout dans le cas des remblais de haute résistance. C'est pourquoi il est souhaitable de mettre au point des remblais et des matériaux de consolidation alternatifs à moindre coût.

Les objectifs visés par le projet étaient de

1.déterminer les avantages relatifs des divers remblais (e.g. remblai de haute élasticité) procurant la stabilité nécessaire pour l'exploitation d'un corps minéralisé massif.

2.établir les spécifications techniques d'une gamme de remblais de haute qualité, susceptibles d'être mis en place à des coûts réduits, et adaptés à l'exploitation en masse de gisements ayant une haute teneur en minerai.

Afin de placer les recherches dans leur cadre, nous passons en revue les principes de l'ingénierie des remblais souterrains, ainsi que quelques méthodes conceptuelles simples.

Nous avons visité des mines diverses en Ontario et au Québec pour connaître l'état des pratiques courantes du remblayage souterrain dans les mines au Canada.

On présente une brève description du remblayage hydraulique conventionnel et de la théorie des remblais rocheux, avec une discussion des aspects pratiques du transport, de la mise en place et du drainage. On énumère les caractéristiques techniques typiques de ces remblais.

On a procédé à un survol des publications traitant de quelques techniques rares et innovatrices, y compris l'examen des remblais à pâtes et du remblai hydraulique à haute densité, ces derniers se trouvant à la pointe du progrès des techniques de remblai à partir de résidus de concentration, ainsi que les systèmes pneumatiques de remblayage et de congélation.

Grâce aux termes d'un sous-contrat du projet, nous nous sommes servis d'un centrifugeur expérimental de l'Université Queen's afin d'élaborer les spécifications conceptuelles de plusieurs techniques innovatrices de remblayage souterrain susceptibles de réduire les frais de cette opération. Parmi les techniques examinées, mentionnons le renforcement du remblai au moyen de géotextiles, ainsi que le remblayage par coulées successives, ou avec une paroi laissée en pente. Il ressort de l'étude des essais modélisés que de telles techniques permettraient d'obtenir des améliorations considérables par rapport aux techniques présentement utilisées.

Nous avons également examiné le potentiel des systèmes de remblayage monolithique dont on se sert généralement dans les mines de charbon ou de roche tendre. Grâce aux termes d'un sous-contrat relevant du projet, nous avons assujetti à une longue série d'épreuves en laboratoire divers matériaux utilisés dans le remblayage monolithique. Une gamme de caractéristiques techniques a été déterminée, et des mélanges optima ont été identifiés. Ensuite, dans le cadre d'un essai sur place, nous avons utilisé les constituants compactés du remblayage monolithique pour en faire un mur de soutènement. Des matériaux analogues ont servi à obturer une taille remblayée.

L'instrumentation in situ, au moyen de laquelle on soumet les remblais et le massif rocheux à un contrôle continu, joue un rôle crucial pour notre compréhension du comportement dynamique des remblais dans le milieu souterrain. Dans le présent rapport, les résultats des contrôles effectués dans le cadre du projet sont présentés sous forme d'une série d'études de cas. Nous y ajoutons certaines études de cas déjà effectuées par Falconbridge et Kidd Creek Mines en utilisant le même procédé.

Le travail considérable fait en laboratoire consistait à mettre à l'épreuve les liants alternatifs aux remblais hydrauliques ou rocheux. Nous présentons les résultats pour les liants suivants: laitier ferreux broyé, cendre volante, laitier de cuivre broyé, anhydrite. Nous avons examiné les divers procédés pour accélérer la prise de ces matériaux pouzzolaniques et cimentaires.

CONCLUSIONS DU PROJET ET RECOMMANDATIONS.

La réalisation du projet de Remblais alternatifs pour les mines de l'Ontario représente 5 années de travail qui comprennent un examen approfondi de la documentation, un tour d'horizon des opérations de remblayage pratiquées dans plusieurs mines en Ontario et au Québec, une série complète d'essais en laboratoire étudiant les diverses substitutions possibles, la mise à l'épreuve de ceux-ci sur le terrain et la mise en pratique de l'instrumentation couramment appliquée au remblayage. Ce travail a révélé plusieurs façons d'améliorer les opérations de remblayage, en indiquant en même temps les voies dans lesquelles les recherches devront s'engager avec les meilleures chances de succès.

Les liants forment une grosse part du prix de l'amenagement de remblais. C'est pourquoi ce projet insiste sur la recherche et l'analyse de solutions alternatives concernant les liants. Une quantité considérable d'analyses en laboratoire sur les agents chimiques a été effectuée, en même temps qu'une enquête portant sur les moyens d'obtenir des performances optimales à partir de ces possibilités. Au cours du projet, les matériaux suivants ont été envisagés comme liants possibles des résidus hydrauliques ou du remblai rocheux.

1. Ciment portland ordinaire. Le liant industriel standard utilisé comme norme de comparaison pour évaluer les autres.

2. Laitier ferreux broyé. Différents laitiers ont été analysés en tant que liant, à la fois pour remblai hydraulique et pour remblai rocheux. Une différence importante d'efficacité d'un laitier à l'autre a été remarquée. Le problème principal avec l'utilisation de laitiers ferreux est la lenteur de la maturation. A la suite de l'opération effectuée sur le terrain, Falconbridge a beaucoup utilisé le laitier ferreux comme liant des remblais hydrauliques à l'exploitation de Sudbury. Le laitier ferreux est beaucoup utilisé dans d'autres mines de l'Ontario, car on peut l'obtenir à bien meilleur marché que le ciment portland.

3. La cendre volante, produit-déchet des centrales électriques à charbon, a été analysée et s'avère un liant qui convient à la fois aux remblais hydrauliques ou rocheux. Deux sortes de cendres ont été étudiées, le type C qui a donné les meilleurs résultats et le type F qui ne s'est pas révélé aussi bon. La cendre volante de type C est utilisée en mélange avec le ciment portland pour faire un liant qui revient le moins cher dans tous les cas. (Une teneur de 60% est utilisée avec succès). La stabilité d'un remblai consolidé au moyen d'un liant avec adjonction de cendre volante peut dépasser, à moindre coût, les résultats obtenus avec du

ciment portland, mais là aussi la vitesse de développement de sa résistance est moindre. Les mines Kidd Creek, ainsi que de nombreuses autres mines en Ontario et au Québec, utilisent des liants additionnés de cendre volante pour la consolidation des remblais rocheux.

4. Le laitier de cuivre granulaire provenant de Kidd Creek a été étudié pour son potentiel en tant que liant de remblai. Le travail d'analyse a indiqué qu'on pouvait l'utiliser avec succès en le mélangeant à du ciment portland en proportions allant jusqu'à environ 40%. Les coûts de broyage du laitier, pour qu'il soit à la grosseur voulue pour obtenir une réactivité suffisante n'encouragent pas son utilisation, surtout si l'on en compare le prix avec ceux du laitier ferreux et de la cendre volante. Malgré cela, la description du travail d'analyse peut servir à d'autres exploitants se lançant dans des enquêtes sur le potentiel de leurs laitiers.

5. L'anhydrite a été examiné au laboratoire comme liant dans les remblais hydrauliques composés de résidus de concentrateur. Il s'est révélé être un liant de qualité capable de produire un remblai de grande résistance, mais les adjonctions pratiquées au cours des essais étaient irréalisables dans la pratique. Ce travail de recherche n'est que d'intérêt théorique à ce stade; un travail supplémentaire utilisant des additions réalistes est nécessaire.

L'adjonction d'anhydrite, de gypse et de chaux aux liants constitués de ciment portland mélangé au laitier de cuivre ou à la cendre volante a permis l'accroissement de la résistance à la compression du remblai. Les augmentations étaient souvent importantes, le meilleur résultat étant obtenu en ajoutant 3% d'anhydrite à un mélange de laitiers, ce qui a provoqué un accroissement de résistance de 217% et 240% après 28 et 56 jours de séchage. La possibilité d'améliorer la qualité des laitiers de cuivre et de cendre volante en y ajoutant de petites quantités d'anhydrite ou de gypse est une conclusion importante du projet.

On a fait des essais avec différents accélérateurs chimiques généralement utilisés dans l'industrie du béton avec des liants au laitier de cuivre mélangé à du ciment portland. Les résultats se présentent comme suit:

1. Daracell, produit par W.R. Grace Chemical Company, a donné de bons résultats avec le liant de ciment portland mélangé à du laitier de cuivre. Pozzolith 133-HE, produit par Master Builder Chemical Company, a donné lui aussi de bons résultats avec le même mélange de ciment portland et laitier de cuivre. Cependant, le prix élevé de Daracell et de Pozzolith rend difficile l'utilisation à l'échelle industrielle de ces produits.

2. K_2SO_4 a un bon potentiel. Une addition de 5% de K_2SO_4 a augmenté de 40% la résistance du ciment portland avec laitier de cuivre après 50 jours de séchage.

3. L'accélérateur courant ($CaCl_2$) utilisé dans le bétonnage a un bon potentiel. Bien que l'augmentation de résistance obtenue par ce produit chimique ne soit pas aussi spectaculaire que pour les autres produits examinés, son prix modéré en fait un adjuvant potentiellement utilisable. Le travail d'analyse a révélé que 2% de $CaCl_2$ augmentait de 20% la résistance initiale d'un mélange de laitier de cuivre et de ciment portland, ce qui indique que le $CaCl_2$ pourrait être utilisé si un cycle d'exploitation minière plus rapide était nécessaire.

La conclusion des résultats ci-avant mentionnés concernant les accélérateurs et le laitier de cuivre est d'importance. Les mines employant comme liant de remblai les laitiers ferreux les plus communément utilisés pourraient considérer les résultats de l'utilisation de ces accélérateurs comme point de départ de recherches portant sur l'amélioration du rendement de leur propre produit.

Il est reconnu que, en pratiquant des méthodes innovatrices, il est possible d'améliorer la performance mécanique des remblais consolidés dans le cas de la récupération des piliers, où une surface du remblai est exposée. Une modélisation au centrifugeur a été utilisée pour étudier trois de ces innovations employant des remblais hydrauliques de résidus cimentés.

1. Paroi de remblai en pente. En creusant la taille avec des parois en pente, le remblai peut être disposé de façon à ce que au moins une partie de sa surface libre soit en pente. Des travaux-pilote montrent qu'une telle paroi de remblai a plus de deux fois la stabilité prévue par la théorie, et plus de trois fois la stabilité prévue pour une paroi de remblai verticale.

2. Paroi de remblai à haute densité. Quand la possibilité existe, il serait avantageux de disposer une paroi de remblai de grande robustesse contre le pilier à extraire. Une fois dégagée, cette paroi renforcée serait plus stable que si la même quantité de liant avait été dispersée dans un remblai uniforme. Les essais indiquent qu'une paroi de remblai à haute densité peut soutenir et retenir un remblai non-consolidé, ce qui pourrait avoir pour conséquence d'importantes économies de liant.

3. Remblai en couches superposées. Cette conception découle de l'expérience de l'exploitation par chambres remblayées: des couches denses de remblai à forte concentration en ciment sont disposées en planchers de manoeuvre au-dessus de couches de remblai peu consolidé.

L'extraction de piliers au voisinage des chantiers ainsi remblayés montre que la surface exposée d'un tel remblai est remarquablement stable. Les travaux-pilote montrent que le remblayage en coulées successives est grandement supérieur à une mise en place non différenciée. Une augmentation considérable des hauteurs de surface exposée est possible avec la même consommation de liant et sans perte de stabilité du remblai.

Les techniques de modélisation au centrifugeur ont été mises à contribution pour étudier les possibilités d'utilisation des grillages géotextiles et autres éléments de renforcement mécanique des remblais hydrauliques de résidus. De ces analyses on a pu constater que les armatures à mailles pourraient effectivement renforcer les remblais, mais que les quantités nécessaires sont si élevées que leur utilisation n'est pas économique. Le renforcement procuré par les grillages géotextiles était très satisfaisant, et les problèmes pratiques de leur installation méritent d'être examinés.

Une conclusion importante de ce rapport est que la modélisation au centrifugeur est une méthode très utile pour l'élaboration de modèles physiques dans la conception des remblais.

Parmi les essais effectués au cours de ce projet, nous avons examiné le remblayage monolithique et plusieurs agents de cimentation, méthodes empruntées aux mines de roche tendre, qui en utilisent avec succès d'importantes quantités. Il est établi que ces matériaux ont actuellement des applications limitées dans l'industrie de la roche dure. Les petites quantités nécessaires rendent très onéreuses les applications que l'on a essayées ici. Les problèmes d'ordre pratique rencontrés avec l'emploi de petits lots et l'absence de personnel expérimenté pourraient être surmontés à condition de trouver une application susceptible de bénéficier des économies d'échelle.

Une vaste campagne de monitorage et d'instrumentation des chantiers d'abattage a fait partie du projet. Comme on s'y attendait, les résultats ont montré que la paroi adjacente à l'excavation se dilate, tandis que la paroi éloignée de l'ouverture se contracte. Après chaque tir, l'expansion continuait dans toute la paroi adjacente à la surface dynamitée. La majeure partie de l'expansion enregistrée a eu lieu après des sautages importants. Une fois le remblai rocheux consolidé mis en place, le mouvement d'expansion de la paroi s'est arrêté et le déplacement dans le sens inverse a atteint jusqu'à 0,9 mm dans le chantier observé. Ceci laisse supposer que le remblai rocheux que l'on a mis en place a provoqué un état de compression active suffisant pour soutenir la paroi détendue du chantier. Le monitorage des effets des tirs sur le remblai rocheux consolidé aux Mines Kidd Creek a apporté les conclusions suivantes. Le coefficient de perméabilité du massif au remblai rocheux consolidé était en moyenne de 37%. On a estimé à 0,41 db/m le coefficient d'absorption du remblai rocheux consolidé. La vélocité de propagation des ondes sonores (ondes P) dans le remblai rocheux consolidé était en moyenne de 3 220 m/s. Une vélocité de particule estimée à 30 cm/s a produit l'effritement d'un remblai rocheux consolidé en bon état.

Les examens indiquaient que l'essai de la charge portante fournirait une méthode rapide et fiable de déterminer sur le terrain la résistance du remblai rocheux consolidé.

Une étude des possibilités autres que les techniques actuelles de mise en place du remblai hydraulique a été faite. Des systèmes employant les remblais à ciment pur (sous forme de pâte) et des remblais hydrauliques à haute densité ont pour but de minimiser la quantité d'eau de transport nécessaire, ce qui permet de concevoir un remblai qui réponde aux critères de résistance et de fonctionnement nécessaires au rendement maximum sur le terrain, sans compromettre les exigences de transport et de drainage. La technologie favorisant les remblais à faible teneur en eau est assez bien comprise par un nombre restreint d'exploitants dans le monde. La diffusion du savoir et la reconnaissance des applications éventuelles sont nécessaires pour que l'industrie profite de ces nouvelles alternatives en matière de remblais.

RECOMMANDATIONS

Les recommandations suivantes découlent du travail réalisé au cours de ce projet.

1. L'instrumentation des remblais est toujours difficile et souvent onéreuse. Un effort est nécessaire pour développer davantage la fiabilité et la rentabilité des essais. Ceci encouragera davantage les exploitants à procéder à des prélèvements d'échantillons réguliers et à l'instrumentation. Avec le temps, on établira des bases de données sur le comportement des remblais et leurs caractéristiques sur le terrain pour obtenir de meilleurs procédés à l'avenir.

2. Il faut réviser régulièrement les pratiques courantes de remblayage minier au Canada. Un rapport à ce sujet doit être diffusé au sein de l'industrie, ce qui fournira des critères pour la conception et la mise au point des remblais. Un examen sélectionné et moins fréquent du remblayage pratiqué à l'étranger doit se faire pour repérer les tendances et le développement des nouvelles technologies.

3. La recherche portant sur les effets de la ségrégation sur la résistance globale des remblais hydrauliques et des remblais rocheux est recommandée. La ségrégation est observée dans tous les types de remblais et, dans certaines conditions, peut jouer un rôle plus déterminant

dans la résistance du volume total d'un remblai à surface exposée que la composition du liant. Les opérations optimales de remblayage exigent une meilleure compréhension des effets autant négatifs que positifs, de la ségrégation.

4. Nous recommandons les enquêtes portant sur le potentiel du remblai hydraulique à haute densité comme variante technologique moins avancée pour remplacer les systèmes nécessitant l'adjonction de pâtes. En associant le remblai hydraulique à haute densité à la technique des surfaces libres en pente ou à celle du remblayage en coulées successives selon la modélisation effectuée dans ce projet, et en utilisant un liant bon marché de résistance supérieure, il est possible d'obtenir des remblais ayant des résistances égales ou supérieures à celles qu'il est possible d'obtenir avec les remblais à pâtes, sans les inconvénients technologiques causés par la pâte.

5. Le remblai rocheux est plus répandu depuis que les exigences de résistance et de rigidité augmentent en fonction des changements dans les méthodes et de l'augmentation des profondeurs d'exploitation minière. Il est conseillé d'accorder plus d'importance à la recherche dans le domaine de la technologie des remblais rocheux. Cela doit comprendre des études granulométriques sur la taille optimale des composantes d'un remblai rocheux.

6. Il faut étudier le comportement d'un remblai confiné posé comme remblai rigide de façon à limiter la convergence des parois du chantier d'abattage et prévenir les coups de terrain, tout en étudiant l'influence des liants et des dimensions granulaires sur la rigidité dans le confinement. Beaucoup de recherche a été effectuée sur le calcul des propriétés technologiques nécessaires au remblai pour que la surface exposée soit stable pendant la récupération des piliers; mais avec la demande croissante de remblai rigide, il est nécessaire de mieux comprendre les caractéristiques des remblais confinés.

7. Le laitier ferreux et la cendre volante sont maintenant bien établis comme substituts rentables remplaçant le ciment portland comme liant dans les remblais hydrauliques et rocheux des mines de l'Ontario. Pour le moment, il semble inutile de rechercher encore d'autres substituts comme les laitiers non-ferreux. Le choix d'un liant de remblai est dorénavant fonction des conditions géotechniques et commerciales particulières et ne dépend pas de recherches plus approfondies.

8. Nous recommandons des enquêtes plus poussées sur le potentiel des produits de type Tekfill. Ces produits sont utilisés de façon courante dans les exploitations minières d'Afrique

du Sud, conjointement avec les remblais hydrauliques denses. L'emploi de produits analogues pourrait servir à développer des remblais hydrauliques plus denses pour les mines de l'Ontario.

9. Il faut examiner davantage l'application de la technologie du remblayage monolithique à l'exploitation des gisements métallifères en roche dure. Les résultats de ce projet montrent que cette technologie habituellement utilisée dans les exploitations minières de charbon et de roche tendre, présente un potentiel considérable pour des applications spécialisées dans les mines de l'Ontario.

10. Le projet indique qu'il y a un potentiel prometteur dans l'emploi d'agents chimiques modifiant la réactivité et les temps de séchage des liants de remblai. Nous recommandons des examens plus poussés dans ce sens de façon à mieux déterminer les répercussions économiques de l'emploi de divers agents chimiques.

11. La plupart des instruments utilisés pour la détermination in situ des caractéristiques du remblai avaient été développés pour leur usage avec les roches beaucoup moins sujettes à la perturbation. Il faut se pencher sur la façon correcte d'installer et de disposer les instruments de mesure, afin d'assurer leur fiabilité.

EXONÉRATION DE RESPONSABILITÉ

Les résultats présentés dans le présent rapport se rapportent exclusivement aux matériaux soumis à des essais dans le cadre d'un projet de Recherche et Développement. Falconbridge Limited ne privilégie aucun des produits mentionnés dans ce rapport.

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11. Concrete slump test

I. Instruments for in situ measurement of fill properties

J. An investigation into the design of tight packs in hard rock mine backfill.

CHAPTER 1

INTRODUCTION

1.1 Project Objectives

1.2 Project Research Work

1.3 Terminology

1.4 Backfill Historical Development

1.5 The Role of Backfill

1.6 Backfill Binder Alternatives

1.7 Structure of the Report

1.1 PROJECT OBJECTIVES

This report documents the results of research work carried out by Falconbridge Limited's Sudbury Operations as principal contractor, and its sub contractors, Kidd Creek Mines Limited, Kingston GeoResearch, McGill University, Trow Ontario Ltd, and Infotech Research as part of the project "In situ properties of backfill alternatives in Ontario mines", a project conducted under the Canada/Ontario Mineral Development Agreement for The Department of Supply and Services, contract number DSS# 09-SQ.23440-6-9011.

The objectives of the project were;

1. To determine the relative merits of various backfilling alternatives, including high modulus fill, to underground stability in mining a massive ore deposit.
2. To establish general engineering specifications for a range of high quality, low placement cost backfills applicable to the bulk mining of rich orebodies.

The project contract outlines the statement of work for the contractors as;

1. Select suitable test sites.
2. Propose and monitor the placement for various backfills. The candidate fills should encompass a wide range of aggregate-binder combinations and strategic (possibly novel) reinforcement techniques.
3. Install instrumentation around mining panels and within the fill and immediate ore/waste rock confines of test sites. The instrumentation should provide measurements of the rock mass and fill deformations as mining, filling, and ultimate curing progress.
4. Monitor all instruments and perform additional tests sufficient to characterize each fill's performance in relative terms.
5. Establish recommendations for and engineering specifications of backfill alternatives.

A primary goal of the project was to investigate ways of reducing the cost and improving the performance of the common backfill systems, conventional hydraulic fill and rockfill. Extensive test work was conducted, which centered on how to reduce the cost of consolidated backfills by using alternative cement binders or chemical additives.

Innovative backfill alternatives were also investigated. Monolithic packing systems commonly used in soft rock mining operations were reviewed for their potential application in Ontario hard rock mines. These materials were tried for use in stope capping and fill fence construction.

A literature review of paste, high density hydraulic fills, frozen fill and pneumatic fills was made. No laboratory or trial work was conducted on these innovations. Paste and high density fills show great promise as backfill alternatives and they are actively being researched by other organizations.

A series of centrifuge model studies was conducted to investigate innovations in conventional hydraulic fill placement. These were the physical reinforcement of hydraulic backfill with geotextile materials, improvement in fill wall stability by using high strength fill at the wall with cheaper low strength fill behind, and sloping the fill wall to improve stability. These studies have helped prove the usefulness of centrifugalmodelling.

The project involved some extensive monitoring and instrumentation of backfill and the rock surrounding backfilled voids.

As background to all the research noted above, this report contains some general chapters which simply describe the theory and practice of current backfill operations. Also, for completeness a short review of the important physical parameters of backfill has been included. These general chapters cover the subjects only in a superficial manner. A number of papers and texts are available which give a more comprehensive review of the theory.

1.2 PROJECT RESEARCH WORK

Table 1.1 below gives a summary of the individual research projects carried out as part of the overall project. Much of the research work concentrated on minimizing costs and maximizing performance of binders used in backfill. This is the area of greatest potential return on the research dollar.

SUBJECT	DESCRIPTION OF TEST WORK	SUBCONTRACTOR
Kidd Creek Mines copper slag as a binder.	Test for pozolanic activity in the presence of portland cement.	Kidd Creek Mines
	Effect of Blaine size on performance.	Kidd Creek Mines
	Strength development with varying degrees of slag replacement in portland cement.	Kidd Creek Mines
	Effect of requenching and lime enrichment on slag performance as a binder.	Kidd Creek Mines
	Effect of acidulation on slag performance as a binder.	Kidd Creek Mines
	Effect of gypsum and lime addition on slag performance as a binder.	Kidd Creek Mines
	Effect of chemical accelerators on slag early strength development.	Kidd Creek Mines
	Grindability of slag.	Kidd Creek Mines
Ground Blast furnace slag as a binder	Comparitive performance of different ferrous slags	Falconbridge
	Activation of ferrous slag with lime	Falconbridge
	Activation of blended portland and ferrous slag with lime	Falconbridge
	Underground field trial of ferrous slag as a binder	Falconbridge
	Economics of ferrous slag usage in hydraulic fill	Falconbridge
	Test performance of various slags in rockfill as a binder.	Kidd Creek Mines
Flyash as a binder	Test performance of type F and type C flyash as a binder in rockfill at various replacement rates.	Kidd Creek Mines
	Effect of acidulation on performance of type F flyash as a binder.	Kidd Creek Mines
	Effect of addition of gypsum, lime and sodium silicate on the early strength developement.	Kidd Creek Mines
	Effect of calcium oxide enrichment on strength development of type F flyash.	Kidd Creek Mines
	Test performance of flyash as binder in hydraulic fill.	Falconbridge
Instrumentation	USBM sonic probe to measure fill wall profiles during pillar extraction.	Falconbridge

	Underground trial of pressuremeter for insitu fill strength determination.	Kidd Creek Mines
	Instrumentation of a stope to determine the effect of rockfill on pillar stability.	Kidd Creek Mines
	Instrumentation of consolidated fill to determine blasting damage.	Falconbridge
Modelling	Physical centrifuge modelling of layered fill.	Kingston GeoResearch
	Physical centrifuge modelling of sloped and dense wall fill designs.	Kingston GeoResearch
	Physical centrifuge modelling of fibre and mesh reinforcement of fill.	Kingston GeoResearch
General	Effect of water quality on strength of consolidated fill.	Kidd Creek Mines
	Method of stope capping with cementitious foam.	Kidd Creek Mines
	Field trial of layered fill.	Falconbridge
	Field trial of sloped fill wall.	Falconbridge
	Review of monolithic packing materials.	McGillUniversity
	Effect of chemical additives to improve early strength and setting characteristics of hyd. fill.	Falconbridge
	Field trial of monolithic packing material used to build a fill containment barricade.	Falconbridge
	Trials with chemicals to aid tight filling, lab scale tests.	Falconbridge
	Underground field trial of chemicals to aid tight filling.	Falconbridge

Table 1.1, Research projects undertaken as part of DSS File No. 09SQ.23440-6-9011

Backfill Alternatives in Ontario Mines

1.3 TERMINOLOGY

A common terminology has been adopted for this report.

Hydraulic fills are all those which use water as their transport media. Included in this group are the conventional hydraulic fills common throughout the industry, paste fills which contain such little water content that they appear like toothpaste, and high density hydraulic fills which are somewhere in between these two.

A rockfill is composed of waste rock or other coarse aggregate. Rockfills are transported by bulk materials handling systems, e.g. raises, trucks, conveyors.

Pneumatic fills are those which use air as the transport media.

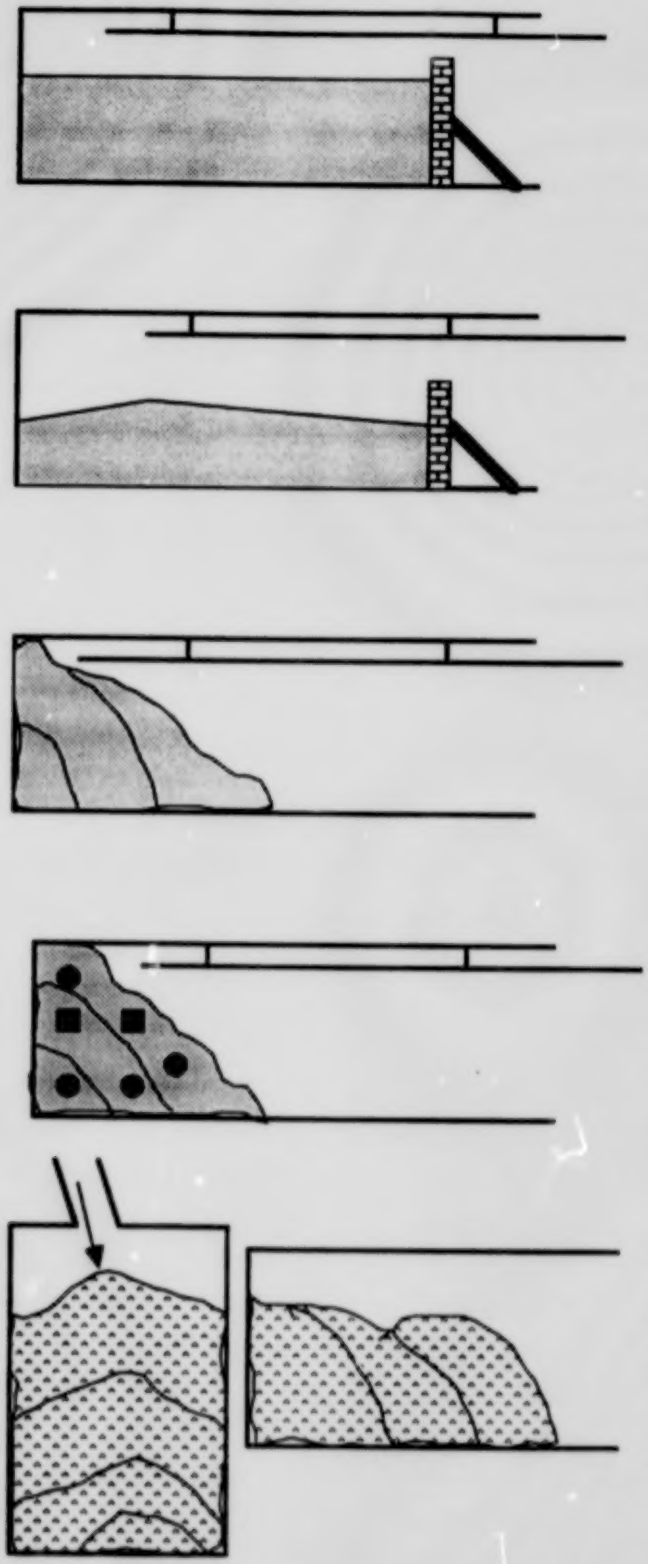
A fill is called consolidated if a cement has been added to give some true cohesion between particles. Unconsolidated fills contain no cement. Portland cement, the most common of binders is often referred to a simply "cement". To avoid confusing ground slags and flyash with "cement" the term binder is used to describe all cementitious materials.

A frozen fill is one which relies upon ice to act as a binder.

Monolithic packing and other cementitious foams are commonly employed in the coal mining industry but have potential for some application in underground hard rock mining. They are based upon quick setting hydraulic mixtures of cements, fly ash, and/or anhydrate which when mixed form a strong cemented mass.

A hydraulic fill may be referred to by its particular material composition. Tailings fill is hydraulic fill composed primarily of mill tailings, similarly sand fill refers to hydraulic fill composed of coarser sands. Other variations on fill terminology are defined in the text.

The distinction between fill types, particularly conventional hydraulic fill and paste type fills is often confusing. Figure 1.1 gives the definitions of fill as used at Falconbridge Limited - Sudbury Operations. These have been used throughout the report.



Conventional Hydraulic Fill

- 60 to 72% solids by weight
- requires fill barricades
- excess transport water to drain
- permeability critical, requires classified material
- can fill to within 0.3 to 0.6m of the backs
- slight natural angle of repose
- average strength achieved
- very high water binder ratio

High Density Hydaulic Fill

- 70 to 78% solids by weight
- requires fill barricade
- little excess transport water to drain
- permeability not critical, unclassified material
- can fill to within 0.3 to 0.6m of the backs
- 10° angle of repose
- up to twice average strength
- moderate water binder ratio

Paste Fill

- 78 to 82% solids by weight
- no fill barricade required
- no drain water
- permeability not critical, unclassified material
- can tight fill to the backs
- up to 65° angle of repose
- greater than twice the average strength
- good water binder ratio

Dense Paste Fill

- 80 to 84% solids by weight
- unclassified tails with coarse aggregate
- no fill barricade
- no drain water
- permeability not critical
- can tight fill
- up to 80° angle of repose
- greater than three times the average strength
- contains up to 50% crushed rock

Rock Fill

- 80 to 92% solids
- mine waste or crushed rock
- no fill barricades
- no drain water
- greater than three times average strength
- less binder required
- good water binder ratio

Figure 1.1, Definition of backfill types, as used at Falconbridge and adopted for this report.

1.4 BACKFILL HISTORICAL DEVELOPMENT

Mine backfilling was first introduced to limit the subsidence above coal mine workings in the Anthracite region of Pennsylvania, USA, in the late 1800's. This original backfilling was by slushing dry waste rock into the mined voids. The practice quickly spread through the coal and into the hard rock mining industries. Problems associated with difficult handling, stability and ventilation when placing the dry fill resulted in the development of hydraulic transport for backfill. The advantages of hydraulic transport soon became apparent and it was being regularly used in coal mines before the end of the 1800's. Hard rock mines began to adopt hydraulic filling in the early 1900's, with the South African mines placing large quantities underground by 1910. By 1927 hydraulic fill was being adapted to the cut and fill method at the Matahambre mine, Cuba. The ability of hydraulic backfill to reduce the amount of timber used for support, hence reducing cost and manpower requirements encouraged its rapid spread through the hard rock mining industry. It remains the most popular backfilling system today.

The addition of cement to hydraulic fill to form a consolidated mass capable of self support over considerable heights was introduced in the 1950's. This opened the way for improved efficiency in the recovery of pillars, but also significantly increased the cost of backfilling. Research into the technology of backfilling and especially into ways of minimizing the cost of cemented fill became of great importance to the mining industry.

The realization that competent consolidated fill could be placed economically to allow extraction of pillars by bulk mining methods encouraged the development of new mining methods and machinery. Advances in geomechanics, drilling and blasting, and the mining of ore at greater depths have in turn placed new demands on the quality and strength of backfill.

Hydraulic fill has not been able to meet some of these demands and this has resulted in a resurgence in the use of rockfill's, now often consolidated by the addition of a binder. Consolidated rockfill was developed to provide a high strength and stiff backfill capable of absorbing ground stress and standing over heights not economic with hydraulic fill.

Recent developments have been the introduction of paste and high density hydraulic fills that use minimum water for transport. They give strengths approaching that achieved with rockfill but still allow the advantages of pipeline transport at low cost.

The coal and soft rock mining industry has moved away from the general use of backfill with changes in mining methods and equipment technology. They do however use specialized backfill materials for road way support and other areas where subsidence may be a problem. The monolithic packing and pneumatic stowing systems may find application in the hard rock mines.

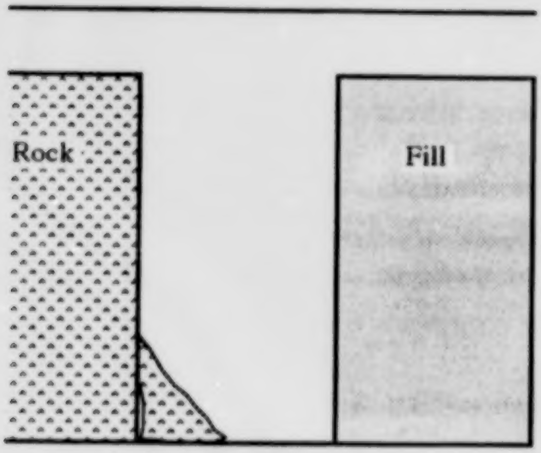
1.5 THE ROLE OF BACKFILL

A backfill may be required to perform in four different ways;

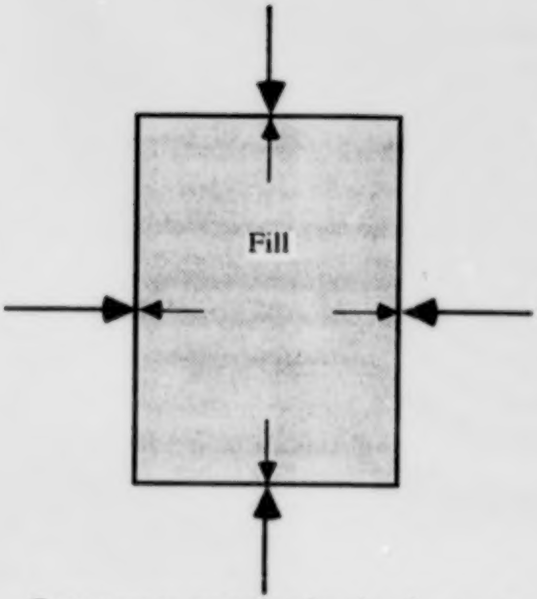
1. A backfill may be required to stand unsupported over significant vertical heights, as in pillar recovery operations.
2. A backfill may be required to resist movement of ground into mined voids providing passive and in some cases active support, improving conditions in adjacent ground.
3. A backfill may be required to provide a competent working floor for mining operations to proceed on top.
4. A backfill may be required to provide a stable roof, perhaps supporting loose material above it, as in undercut and fill mining. Shear strength and compressive strength are important in this role.

A backfill may be required to perform in several of the above roles. Figure 1.2 illustrates the roles of backfill.

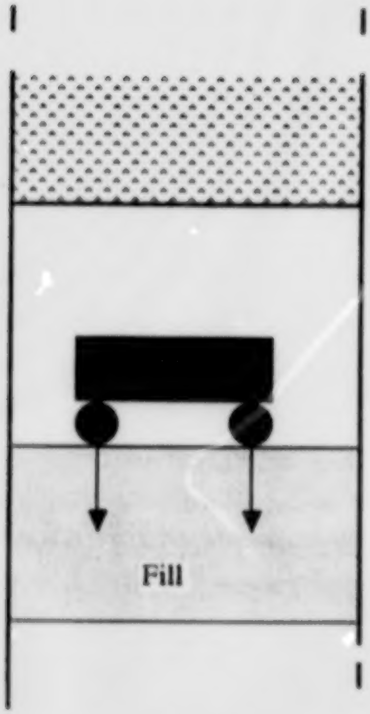
Backfill may also be beneficial for the environment because it allows mine waste to be put back underground. It is also used for long-term prevention of surface subsidence.



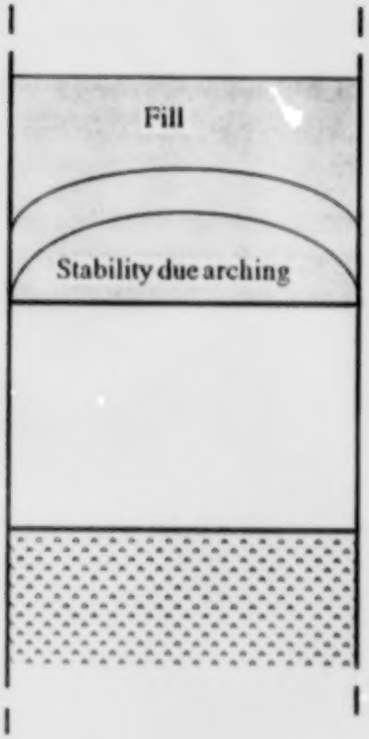
Free standing ability



Convergence control and regional support



Bearing capacity as a floor to support mining activities and help control dilution.


Fill
Stability due arching

Stable roof for undercut and fill mining

Figure 1.2 The roles of backfill.

1.6 BACKFILL BINDER ALTERNATIVES

Portland cement is the standard binder used in consolidated backfills. It is readily available and the technology of its use is well understood. It performs well in most fill types and is the accepted standard against which other binders are judged.

The three important binders used as alternatives to portland cement are;

1. ground blast furnace slag,
2. ground non ferrous slag and
3. fly ash, a by product of the combustion of pulverized coal in thermal power plants.

These alternatives are sometimes available at significant price discounts to portland cement.

Cost of binder is the principal cost in placement of consolidated fills. Hence there is a great incentive to use the lowest possible cost binder which will produce fill of the required strength. Much of the test work in mine backfill research centers on determining the lowest cost binder alternative. This project involved significant amounts of test work on binder alternatives.

Chemical additives and accelerators can be added to binders to modify or improve their performance. The technology of their application is still new but they have shown some good potential. A trial of several chemical additions was conducted as part of the project.

1.7 STRUCTURE OF THE REPORT

This report has attempted to present the results of the diverse research project within the framework of some theory and a discussion on common backfilling practices. The theory is reviewed at a superficial level, with only sufficient depth so the intentions and consequences of the research work may be understood. Chapter 2 contains a short overview of the important engineering properties of backfill and some simple design equations.

Chapter 3 reports the results of visits made to various mines in Ontario and Quebec to investigate existing backfilling operations.

Common practices and practical considerations in rockfilling large stopes and in hydraulic filling are presented in chapters 4 and 5. Chapter 4 is an overview of hydraulic filling operations. Chapter 5 describes rockfilling operations at the Kidd Creek mine, along with the practical design principles used there.

Chapter 6 considers innovative backfill systems. It includes a literature review of paste, frozen, and pneumatic backfill types. Also included in this chapter are summaries of laboratory centrifuge modelling of innovations in hydraulic backfill and of strength testing of monolithic packing.

Results of monitoring and instrumentation of in situ backfill conducted for this project and selected previous work is presented in chapter 7.

Results of investigations into binder alternatives for both hydraulic and rock fills, which comprised a considerable portion of the project work are presented in chapter 8.

Laboratory results and test work conducted by various subcontractors are provided in the appendices.

CHAPTER 2

PHYSICAL PROPERTIES AND STRUCTURAL CHARACTERISTICS OF BACKFILL, AN OVERVIEW

2.1 Introduction

2.2 Important Material Properties of Backfill
  2.2.1 Solids Density
  2.2.2 Particle Size and Distribution
  2.2.3 Particle Shape
  2.2.4 Chemical and Mineralogical Composition

2.3 Mass Properties
  2.3.1 Void Ratio or Porosity
  2.3.2 Moisture Content
  2.3.3 Consolidation Characteristics
  2.3.4 Permeability or Percolation Rate
  2.3.5 Unit Weight and Bulk Density
  2.3.6 Segregation

2.4 Strength and Stiffness
  2.4 .1 Shear Strength
    2.4.1.1 Unconsolidated Backfill
    2.4.1.2 Particle Size and Shape
    2.4.1.3 Porosity or Packing Density
    2.4.1.4 Binder
  2.4.2 Compressive Strength
    2.4.2.1 Binder
    2.4.2.2 Particle Size Distribution
  2.4.3 Stiffness
    2.4.3.1 Confinement
    2.4.3.2 Binder

- 2.4.4 Tensile Strength
- 2.4.5 Bearing Capacity

2.5 Binder Addition

- 2.5.1 Consolidated Fill Strength
- 2.5.2 Design Equations
    - 2.5.2.1 Talbot Curve
    - 2.5.2.2 Hydraulic Fill Equations
    - 2.5.2.3 Rockfill Equations
- 2.5.3 Binders and Time
- 2.5.4 Curing Temperature
- 2.5.5 Chemical Composition

2.6 Conditions of Placement

2.7 Theoretical Backfill Design

- 2.7.1 Failure Modes of Backfill
- 2.7.2 Stability of Free Standing Face of Fill
    - 2.7.2.1 Effect of Fill Arching
    - 2.7.2.2 Assumptions
- 2.7.3 Subsidence and Convergence Control

2.8 Determining the Important Properties

- 2.8.1 Sampling Backfill

2.9 Modelling Backfill

- 2.9.1 Physical Modelling
- 2.9.2 Numerical Modelling

2.1 INTRODUCTION

The properties of backfill which affect its performance are considered here. This chapter is presented as background information and reference for the test work done during the project. It does not present any results or conclusions of the project.

The way in which a backfill performs in place, is a function of its material properties, method and conditions of placement, the addition of a binder, and its mass properties. All these factors are inter related and combine to determine the strength and stiffness parameters of the backfill. A backfill is engineered for its role in the mining operation on the basis of its strength parameters. Figure 2.1 illustrates the relationships which determine the strength of a backfill.

Backfills are composed of particulate materials sometimes cemented together to from a consolidated mass. The strength of such materials is best described by shear strength, which includes a component of strength due to cohesion between particles and another component due the frictional resistance of particles trying to move past each other (friction angle). Most static backfill failures may be analyzed in terms of shear strength. However compressive strengths are easier to measure and hence the strength of a backfill is often be reported in terms of compression rather than shear. This is a valid approach as the uniaxial compressive strength of a test sample is directly related to its shear strength. Samples will fail in shear mode.

There are four basic material properties of backfill;

1. Solids density
2. Particle size and distribution of sizes
3. Particle shape
4. Chemical and mineralogical composition

The above material properties combined with the method and conditions of placement determine the important mass properties of a backfill;

1. Void ratio or porosity
2. Moisture content
3. Consolidation characteristics
4. Permeability
5. Unit weight and bulk density

7. Uniformity or segregation

The conditions of placement mentioned above refers to a diverse range of factors involved in the transport and discharge of backfill materials. Some of the more important of these factors are;

1. The pulp density of hydraulic fills
2. Method of mixing rock and binder when placing rockfill
3. Compaction of the fill
4. Method of placing rockfill - by conveyor or mobile equipment
5. Degree of confinement
6. Relationship of the placement point to the geometry of the void being filled
7. Method and efficiency of drainage in hydraulic fill

If a consolidated fill is to be placed, the binder tends to have the most important impact on the strength of the backfill. A number of binder alternatives are available each with its own characteristics. The performance of a binder is generally influenced in some way by all of the material and mass properties of the backfill mentioned above. In turn, prior to hydration the binder is just another particle in the backfill mass and its presence influences the mass properties of the fill. When the binder begins to hydrate and cement the backfill together, time becomes an important parameter in determining backfill strength.

For engineering of a backfill system there are ten important properties which should be determined;

1. Unconfined Compressive Strength, UCS
2. Friction angle, Ø
3. Cohesion, C
4. Elastic modulus, E
5. Moisture content
6. Unit weight
7. Dry bulk density
8. Porosity
9. Solids density or specific gravity
10. Particle size distribution.

These are the measured engineering properties of backfill, which are a consequence of the interaction between the basic material properties and the placement conditions. Standard procedures for determining the properties of backfill are given in appendix H of the report.

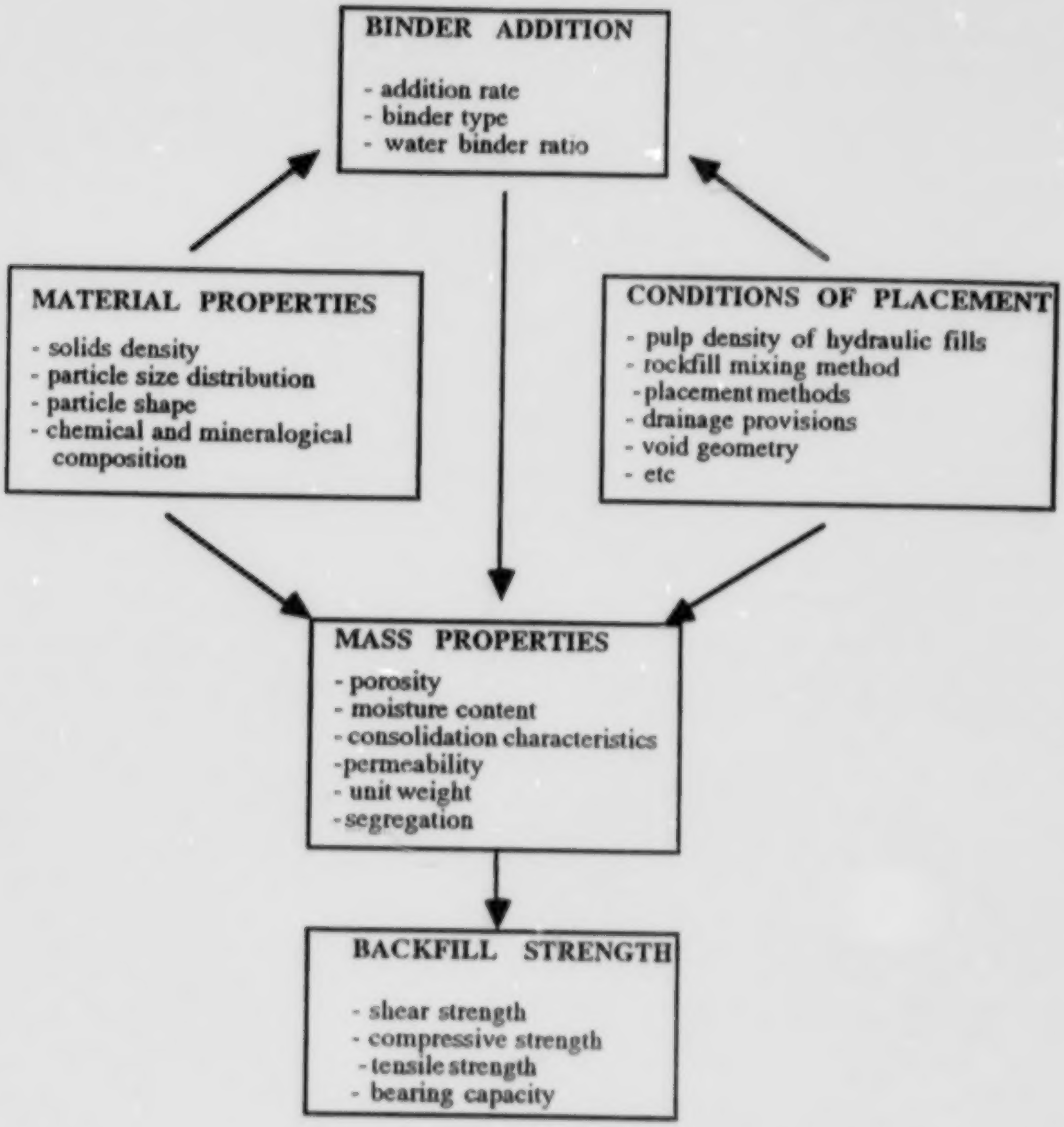

BINDER ADDITION
- addition rate
- binder type
- water binder ratio
MATERIAL PROPERTIES
- solids density
- particle size distribution
- particle shape
- chemical and mineralogical composition
CONDITIONS OF PLACEMENT
- pulp density of hydraulic fills
- rockfill mixing method
- placement methods
- drainage provisions
- void geometry
- etc
MASS PROPERTIES
- porosity
- moisture content
- consolidation characteristics
- permeability
- unit weight
- segregation
BACKFILL STRENGTH
- shear strength
- compressive strength
- tensile strength
- bearing capacity

Figure 2.1, Relationship between fill properties and strength.

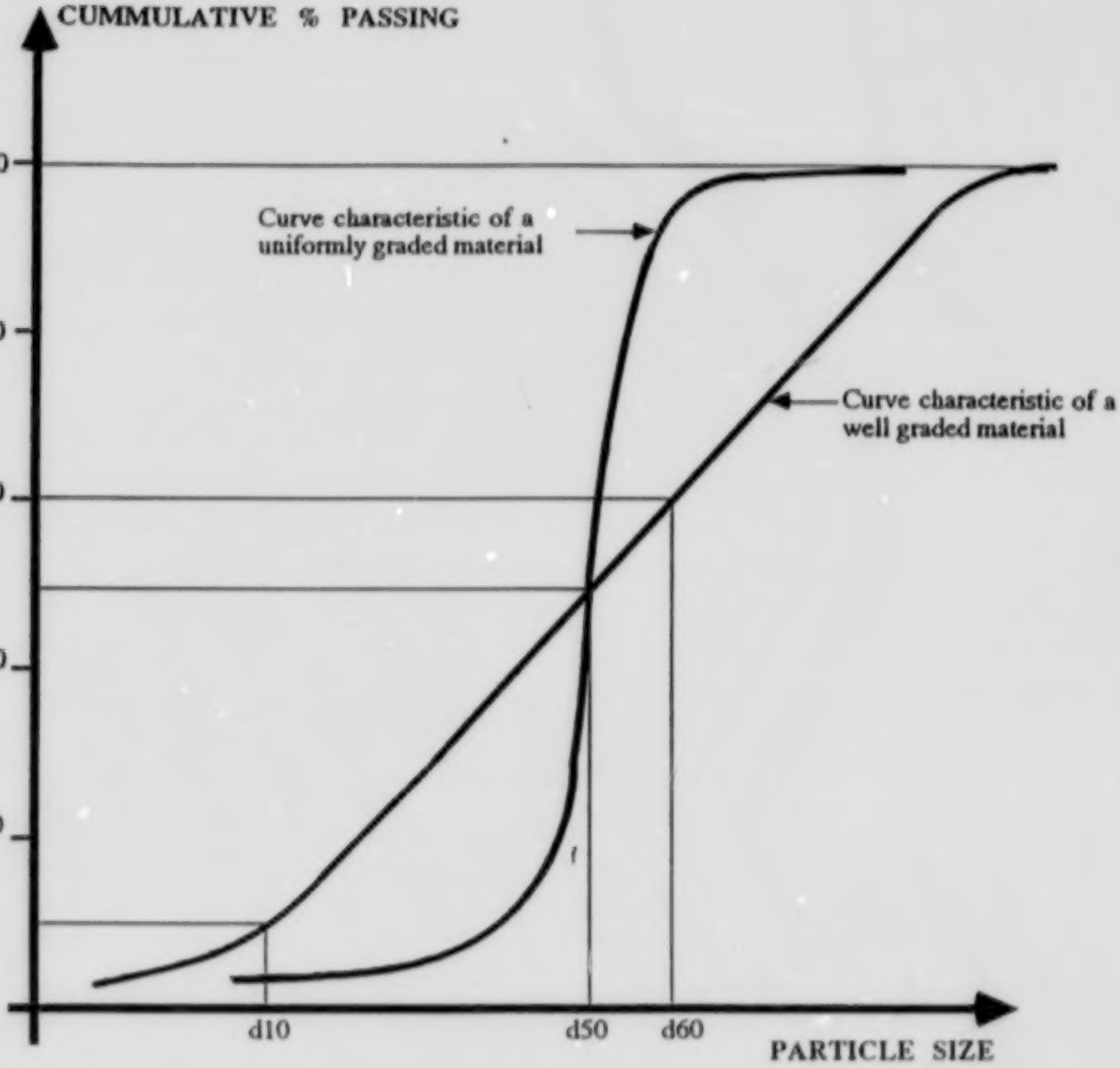


Figure 2.2 Characteristic particle size distributions.

2.2 IMPORTANT MATERIAL PROPERTIES OF BACKFILL

2.2.1 Solids Density or Specific Gravity

The density of the solids making up a backfill is determined by the mineralogical composition. An average density of the different mineral particles is generally considered sufficient for backfill calculations. Solids density is of most concern in hydraulic fill where it is important to the velocity of flow in transport and in the calculation of pulp density (frequently quoted in terms of percent solids by weight). Solids density is one of the factors which determines the rate at which a particle will settle through water.

Solids density is expressed in terms of specific gravity and can be measured with a simple flask and water bath method or by a pycnometer (see appendix H). Typical solids density for backfill materials will range from 2.5 to 3.5.

2.2.2 Particle Size and Distribution

The most important material property which must be determined for a backfill is its size distribution. It affects all aspects of the transport, placement and strength of a backfill, including the performance of the binder in consolidated fill. It is equally important to both hydraulic and rock fills.

A particle size distribution may be described as "well graded" if it contains a wide spread of sizes, or as "uniform" if it contains sizes in only a narrow range. Figure 2.2 illustrates the difference. A unclassified mill tailings would be described as well graded, once classified to remove the finer fractions it would be described as more uniformly graded. In general a well graded material will achieve a greater degree of particle packing and have a lower porosity.

There are standard procedures well documented which specify the method of determining particle size distribution (see appendix H or Thomas et. al. 1979). Several coefficients are used to characterize a size distribution;

1. Uniformity coefficient, $C_u = D_{60}/D_{10}$. D_{60} is 60% passing size (diameter), similarly D_{10} is the 10% passing size. A high value of the uniformity coefficient, C_u indicates a well graded fill, and a low value a uniformly graded fill. A conventional hydraulic fill should have a C_u = 4 to 6.

2. Coefficient of curvature $C_c = D_{30}^2/D_{60} * D_{10}$. This indicates the curvature of the size distribution line. A well graded soil will have C_c = 1 to 3.

3. The value of D_{50} is often used to quickly characterize the average size of a distribution.

The absolute particle size is important to many aspects of backfill, especially when a binder is to be use. One of the principal influences is that absolute particle size determines the surface area of a fill which in turn effects the amount of binder required to coat that area and the moisture content of the fill. Particle size also affects how a fill is transported and placed.

2.2.3 Particle Shape

Particle shape influences the friction between particles in the fill (expressed as the friction angle). Many backfills are composed of particles with an angular shape as a result of blasting or grinding processes. Angular shapes give a high degree of inter particle friction. Alluvial sands used as backfill are more rounded due to weathering and typically have a low angle of friction. Particle shape also influences the natural packing density of a backfill and consequently the void ratio. Rounded particles tend to pack better than irregular angular particles.

2.2.4 Chemical and Mineralogical Composition

The chemical and mineralogical makeup of a backfill is of most importance to consolidated hydraulic fill where some chemical reaction between the binder and fill mass has been known to occur. For this reason, in evaluating a binder for hydraulic fills, especially tailings fills, it is important that the test work be done with the actual fill material. It is generally considered that there will be little, if any chemical interaction between a binder and rockfill because the surface area available for reaction is so much lower.

The quality of the mix water is important to all fill types. It is well known that process water contaminated with oils and other chemicals will adversely affect the strength development in concretes, the same has been observed in backfill placement.

The presence of sulfide minerals in the backfill, particularly finely ground mill tailings, may create some interesting effects. Highly reactive pyrrhotite contained in the tailings of nickel mines of the Sudbury area will react upon exposure to oxygen in the air and may "burn". In these cases the pyrrhotite is removed as a separate tails product and not used as backfill material.

In other cases high sulfide tailings containing large amounts of pyrite and pyrrhotite have been used as "pseudo-binders" because of the cementing action produced when the sulfides oxidize. The oxidization reaction requires water and oxygen and is exothermic. A significant amount of heat can be generated and the oxygen depletion of air in and around stopes filled with such material is possible. If the reaction proceeds too quickly there is potential for a fire hazard to develop and for the generation of noxious fumes when insufficient oxygen is present, (Gignac, 1978). The obvious cost advantages, however have encouraged research into its potential.

2.3 MASS PROPERTIES

2.3.1 Void Ratio or Porosity

Void ratio and porosity are both terms used to describe the same concept, the denseness of the particle packing. This packing has an all pervading influence on the performance of a backfill material.

Void ratio, e, is defined as the ratio of the volume of voids to volume of solids.

$$e = V_v/V_s$$

where; V_v = volume of voids
V_s = volume of solids

Porosity, p, is the percentage of voids in the total volume.

$$P = V_v/(V_v + V_s)$$

Theoretical minimum porosity for uniform spherical particles with perfect packing is 25.9%. In practice with random packing 35% is difficult to achieve and for random shape particles as found in fill, porosity probably varies from 40 to 50%. Values of 42% have been measured for Falconbridge tailings fill (Lausch, 1983).

Uniformly graded fills tend to have a higher porosity than well graded fill. Smaller particles in well graded fill make for better packing and consolidation after placement. A uniformly graded fill with its higher porosity will generally have a lower stiffness and lower strength than a well graded fill. Alluvial sand fills, which often show a characteristic uniformly coarse size grading, are prone to being low stiffness fill with high porosity.

Porosity is also a function of the conditions at placement. A higher water solids ratio hydraulic fill will result in higher porosity in place, i.e. a low pulp density fill slurry results in a high porosity fill. Hydraulic fill should be placed at the maximum practical solids concentration.

Void ratio and porosity are calculated from information obtained when measuring bulk density.

In backfill technology the aim is to keep the fill porosity as low as possible. Low porosity implies good particle contact, hence high friction angle and good strength. Permeability constraints effect how low the porosity can go in a conventional hydraulic fill.

2.3.2 Moisture Content

The amount of water contained in a fill is usually expressed as a percentage of the total weight (as opposed to percentage of the weight of dry solids). It is calculated by determining the mass of a sample of wet material and then the mass after drying in an oven.

$$\text{moisture content} = (\text{wet mass} - \text{dry mass})/(\text{wet mass})$$

Typical moisture content for unconsolidated hydraulic fill with proper drainage is around 12 to 20%. For rockfill the moisture content is typically around 5 to 6%.

Determination of moisture content is important when liquefaction of hydraulic fill is a risk and for calculating the water balance in hydraulic fill pours, e.g. how much water is retained in the fill and how much must be drained and pumped back to surface.

The saturation level or degree of saturation of backfill is defined as the volume of water present as a percentage of the volume of voids. This property is often used instead of the moisture content because it indicates the influence of water on the structure without reference to density (Barrett 1973).

2.3.3 Consolidation Characteristics

The consolidation here is different from that due to the addition of a binder which is how the term is applied throughout the rest of the report. In strict soil mechanics terms, consolidation refers to the slow densification of a soil at a rate controlled by the flow of water out of the soil. This is densification due to particle rearrangement into closer packing geometry as water drains from the voids between particles. Consolidation in these terms mostly concerns hydraulic fills.

The majority of consolidation in hydraulic fills will take place immediately after placement when they are saturated and the particles are settling under self weight expelling water. However the fill will still be capable of significant consolidation after this because of fills characteristically high porosity. If a saturated soil is subjected to some external load the pore water pressure will increase to resist the load preventing any rearrangement of the particles. If the soil is permeable, the increased pore pressure will cause the water to drain from the soil which then gives the particles freedom to take up new positions (better packing). The steady load applied to a fill by closure of stope walls causing fill densification can be called consolidation.

Compaction is considered to be separate process to consolidation. It is the rapid densification of a soil material under the influence of an external load, normally not controlled by the expulsion of water. An unsaturated fill with some voids filled by air, subjected to an external load undergoes compaction. Dry rockfills or drained hydraulic fills with mobile equipment driving across the top undergo compaction.

Consolidation characteristics are important because they will determine the ability of fill to resist deformation of the rock surrounding the filled void. The development of methods for the placement of dense backfill with good consolidation characteristics is a major target of research in backfill technology e.g. paste fills.

2.3.4 Permeability or Percolation Rate

The percolation rate or permeability of water through fill is most often cited as the single most important parameter in the placement of hydraulic backfill. Good permeability is required to ensure that the fill remains fully drained and there is no potential for liquefaction. It is also required to ensure rapid excess water drainage, allowing high placement rates. The size distribution of particles which make up the fill is the principal determinate of the permeability of the fill mass. In general the higher the fines content the lower the permeability. Percolation rate, **Vp**, is defined as;

$$Vp = q / A$$

where:

Vp = percolation rate (mm/hr)

q = flow rate, quantity of water flowing through the fill per hour (mm^3/hr)

A = cross sectional area of flow (mm^2)

The general industry accepted standard for minimum percolation rate of water through fill is 100 mm per hour. At this rate it is generally assumed that there is little chance of fill liquefaction. However this is a somewhat arbitrary limit. Percolation rate is difficult to measure consistently and underground performance may not correspond to laboratory determined values. The rate depends upon water temperature, viscosity, and edge affects. Each operation comes to some suitable compromise based upon practical experience. Laboratory procedures for percolation rate determination at the Falconbridge, Sudbury Operations are included as an appendix to this report.

Permeability, **K**, is an absolute measure independent of water temperature.

$$K = (Q.L.v)/(h.A.w)$$

where;

Q = rate of flow through water through fill sample, m/s

L = distance water must flow, m

v = absolute viscosity of the water, $N.s/m^2$

h = static head, m

A = cross sectional area of flow, m^2

w = unit weight of water, N/m^3

Permeability is the more scientific means of expressing the flow of water through a soil but it is more common to use the easily understood units of percolation rate in backfill operations, so long as one is careful to measure the percolation rate under the same conditions as apply underground. Of particular importance is matching the water temperature; water viscosity changes 30% between 5°C and 20°C.

Liquefaction occurs when the pore water pressure exceeds the normal stress acting on a cohesionless material, creating a mass with no effective shear strength, which is the definition of a fluid. This condition may result from blasting induced vibrations. A liquified fill mass will be a high density fluid which may exert large pressures against containment structures. The potential flow of large volumes of liquified fill into confined underground openings is a major safety concern.

Addition of a binder to hydraulic fill will result in a low permeability material. As the cementing reactions proceed the porosity of the fill will be reduced and the drainage paths will become blocked. The percolation of water through fill will decrease rapidly in a few hours after placement. As the cement content is increased, percolation rate decreases. A mix having a 1:10 ratio of cement to solids by weight may have a percolation rate of 10 mm/hr. Placement of consolidated hydraulic fill presents some particular dewatering problems.

Consolidated fills have true cohesion between particles and are therefore not subject to liquefaction. Permeability of consolidated fills is not important in the long term, once the binder has begun to hydrate and particles become cemented together. It does however affect drainage characteristics at placement.

Hydraulic tailings fills are commonly classified to remove the fine particle size fractions in order to ensure an adequate percolation rate. The size distribution of a fill is the prime determinate of its percolation rate. A uniformly graded, coarse fill with a consequently high porosity will tend to have a high percolation rate while a well graded, low porosity fill will have a low porosity.

It should be recognized that the in situ percolation rate decreases due to stratification, migration of fines and compaction of the fill in the stope. A 20% decrease in void ratio due to compaction of the fill can cause a 50% decrease in percolation rate. The percolation rate also drops with lowering degree of saturation. A partially drained fill from the previous pour will be considerably less permeable. These factors may produce great variation between the laboratory and in situ drainage performance.

2.3.5 Unit Weight and Bulk Density

The unit weight of a fill is needed to calculate failure conditions of exposed faces of backfill. It is a function of the moisture content and the fills dry bulk density. It is measured by simply weighing a known volume of fill or by back calculating from a known dry bulk density and a moisture content. The units are kN/m^3. A typical hydraulic fill will have a unit weight 18 to 24 kN/m^3.The term appears in several of the equations used in backfill design.

Wet bulk density (commonly referred to as just bulk density) is another way of expressing the same concept as unit weight, except the units are kg/m^3. Bulk density units are often more convenient for calculating backfill quantity requirements and to plan for storage, transportation and distribution facilities.

$$1\ kN/m^3 = (1\ kg/m^3 * 9.81\ m/s^2) / 1000$$

2.3.6 Segregation

Segregation or the degree of uniformity is the most difficult of a backfills mass properties to determine. It is also one of the most important in determining the overall strength of a backfill. Segregation refers to the layering and non uniformity which occurs in any backfill, especially conventional hydraulic and rock fills. It is very difficult to observe in the laboratory and just about impossible to quantify. The degree of segregation is determined by observation underground and considered only in a comparative way.

Segregation is the formation of zones of different quality material in a backfill mass. It is determined by the mass properties and conditions of placement - different particles will settle from hydraulic fill slurries at different rates and different sized rocks will behave differently when dumped as rockfill. The mechanisms and effects of segregation are considered further in the chapters which discuss hydraulic and rock fills.

2.4 STRENGTH AND STIFFNESS

Discussions on the properties of backfill above have indicated how much interaction there is between the material and mass properties. What has not yet been considered are the properties of most interest, the backfill strength and stiffness. This is for the good reason that it is necessary to understand strength as a function of many parameters.

2.4.1 Shear Strength

The shear strength of backfill is its most important strength property. A backfill is considered to be a soil like particulate material with its strength dependent upon cohesion between particles and the frictional resistance of particles trying to move past each other while under confinement. The strength of a backfill may be considered as the shear strength expressed by the Mohr-Coulomb equation.

$$S = C + P \tan Ø$$

where;

S = shear strength, kPa
C = cohesion, kPa
P = effective normal stress on the failure plane, kN/m^3
$Ø$ = angle of internal friction, degrees

Shear strength is determined by direct shear box testing, by triaxial compression tests or indirectly from unconfined compression tests. Appendix H describes how shear strength may be measured.

The Mohr failure envelope diagram is used to visualize the two components of shear strength, figure 2.3.

2.4.1.1 Unconsolidated backfill

Unconsolidated backfill will possess shear strength, mostly as a function of the friction between particles, (friction angle). It will also possess a small amount of apparent cohesion which is a function of surface tension in the water between particles (note the difference between apparent and true cohesion). A typical Mohr failure envelope for unconsolidated hydraulic backfill is shown in figure 2.3. The apparent cohesion is an important property of unconsolidated fill. Without it, fill would have a limited ability to support vehicles on top, with it, an unconsolidated fill is given some ability to form a free standing vertical face. At zero moisture content and at saturation point there is no water surface tension and hence no apparent cohesion, somewhere in between these two states is an optimum moisture content with maximum apparent cohesion. The optimum will be a function of particle size, shape and packing density.

2.4.1.2 Particle size and shape

Particle size and shape affect shear strength by altering the friction angle. An angular shaped particle will posses a higher friction angle than a rounded particle (e.g. angular mill tailings fill has a higher friction angle than rounded alluvial sand fills). Larger particles will tend to have a higher friction angle than smaller ones (e.g. rockfill friction angles are generally greater than those of hydraulic fill)

2.4.1.3 Porosity or packing density

As the porosity of a fill decreases the shear strength will improve, figure 2.4. A higher porosity indicates improved particle packing and higher inter particle friction. This trend is a primary reason for the development of high density hydraulic fills which can be placed with a low porosity, hence achieving greater strength. This strength improvement may be used to reduce binder consumption or to form a more competent unconsolidated fill. Attempts have been made to improve fill quality by the use of mechanical vibration and compaction to take advantage of the improvements possible from better particle packing.

Porosity control was discussed earlier. In practice the size distribution of the fill material is the most important design variable. A well graded material will tend to good packing, low porosity and high friction angle.

2.4.1.4 Binder

Addition of binder to fill increases both the cohesion and friction angle components of shear strength. Figure 2.5 illustrates the affect of binder addition on the Mohr failure envelope. The primary action of a binder is to cement particles together providing true cohesion. As the time of curing increases so will the strength of the cohesive bonds and hence the shear strength. In general the relationships between binder and uniaxial compressive strength are true for shear strength.

Binders can increase the friction angle component of shear strength. The fine particles of binder act as a mechanical component of the system creating an overall more well graded fill, with consequently better particle packing. The effect is most noticeable in hydraulic fills and those with a very uniform size distribution. In hydraulic fill we are putting back the fine particles we took out by classification to ensure adequate percolation rates.

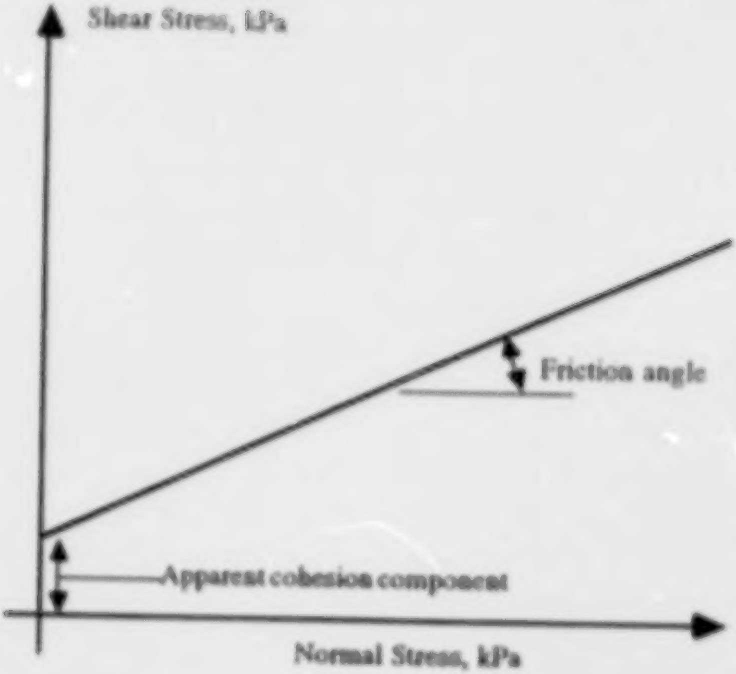

Shear Stress, kPa
Friction angle
Apparent cohesion component
Normal Stress, kPa

Figure 2.3, Mohr failure envelope for unconsolidated backfill.

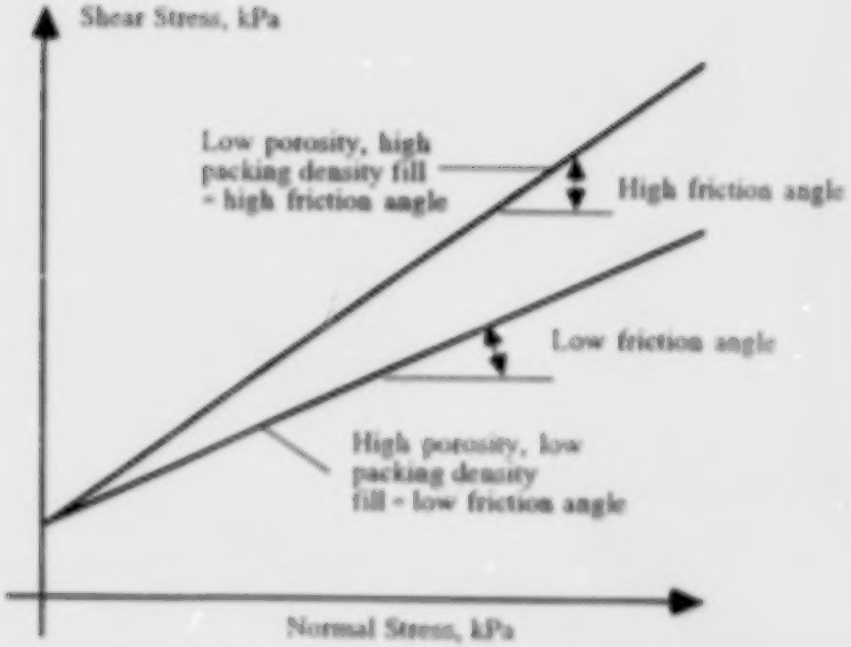

Shear Stress, kPa
Low porosity, high packing density fill = high friction angle
High friction angle
Low friction angle
High porosity, low packing density fill = low friction angle
Normal Stress, kPa

Figure 2.4, Effect of porosity on the shear strength of backfill.

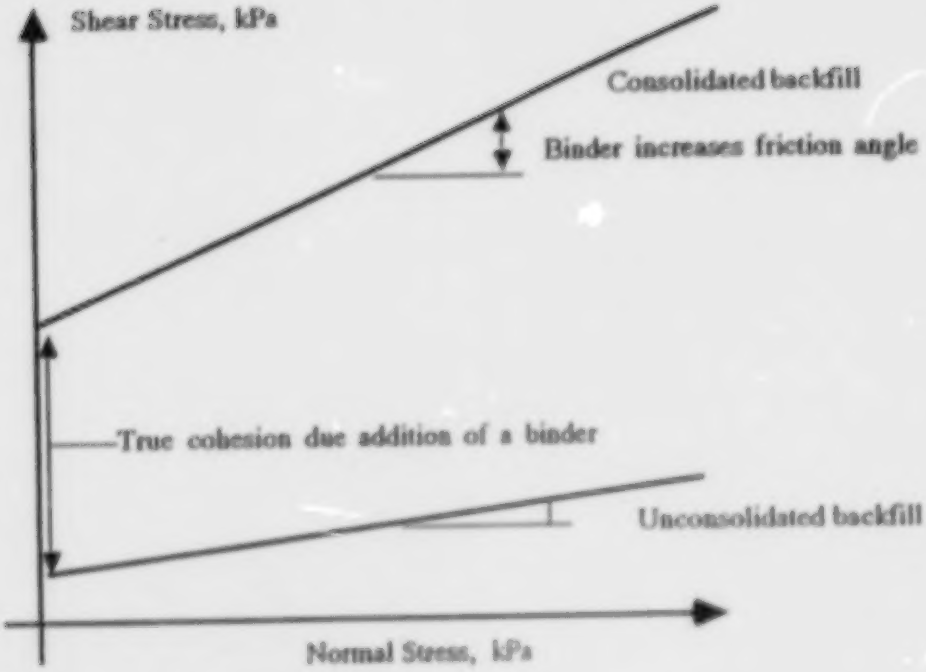

Shear Stress, kPa
Consolidated backfill
Binder increases friction angle
True cohesion due addition of a binder
Unconsolidated backfill
Normal Stress, kPa

Figure 2.5, Effect of a binder on the shear strength of backfill.

2.4.2 Compressive Strength

The compressive strength of backfill is the most commonly measured property. It is usual for unconfined compressive strength, UCS, to be quoted as a measure of fill strength. Compressive strength concerns only consolidated fills. Unconsolidated fills, having no true cohesion possess little compressive strength in the unconfined state. Failure of a backfill test specimen in unconfined compression is by shear. This is also the expected mode of failure of an exposed face of backfill. Hence unconfined compressive strength can be considered a direct measure of the ability of a face to stand vertically.

The popularity of unconfined compressive strength is a consequence of its easy measurement in the laboratory.

UCS is an indirect measure of shear strength. The cohesion and shearing angle of backfill can be obtained indirectly from unconfined compressive strength tests and from failure observation. The internal friction angle can be calculated using the following relation;

$$\text{shearing angle} = 45 + (^1/_2 \text{ internal friction angle})$$

The cohesion component of shear strength is approximately one half to one third of the measured uniaxial compressive strength.

2.4.2.1 Binder

The performance of a binder added to a backfill is usually measured by testing the uniaxial compressive strength. Binder is added to improve the shear strength, however since uniaxial compressive strength is easier to measure, it is the normal standard used. The compressive strength increases with the addition of binder. Figure 2.6 shows the results of laboratory testing of Falconbridge hydraulic tailings fill with increasing amounts of portland cement added. The same type of relationship is seen with rockfills, except the strengths achieved are much higher.

2.4.2.2 Particle size distribution

Just as shear strength is affected by the size distribution of particles in the fill, so compressive strength will reflect the same relationship. A well graded material with good particle packing will show higher compressive strength at the same binder content than a uniformly graded material. The difference can be quite significant and is important in rockfill operations. In hydraulic fill the need to maintain permeability restricts the ability to modify size distribution for optimum strength (an incentive for the development of low water content fills).

Fig 2.6

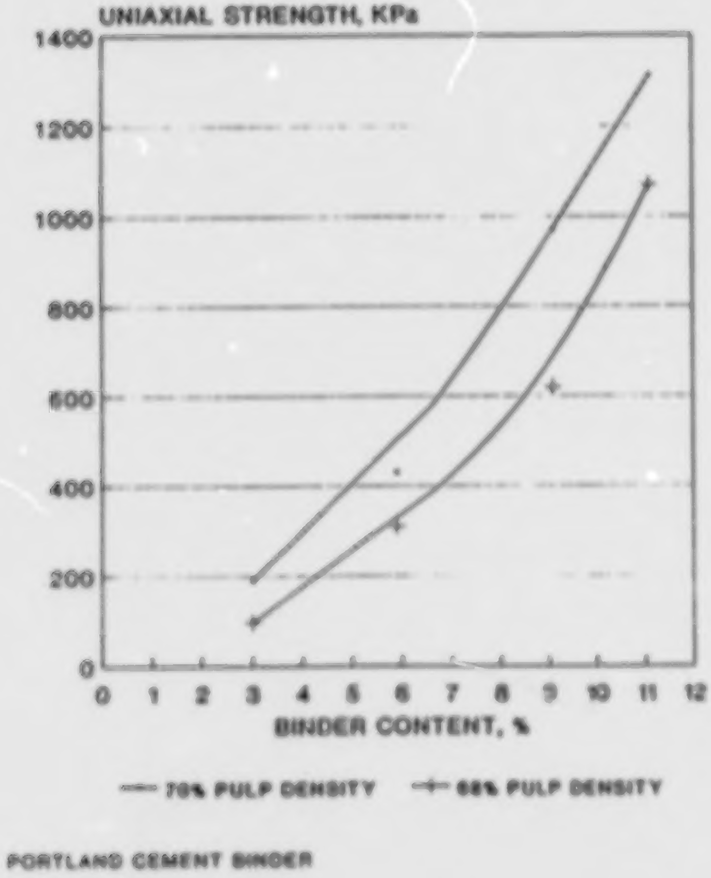


Figure 2.6 Strength of Falconbridge hydraulic tailings fill.

2.4.3 Stiffness

Stiffness is important to fill in its regional support role where it is required to limit ground convergence or to accept some ground load reducing stress in adjacent pillars. Stiffness is expressed as the modulus of elasticity, E.

E = axial stress / axial strain

The stress strain relationships in granular materials are complex, since these materials develop considerable non recoverable strain and often exhibit non linear stress strain behavior. It is probably more valid to consider E as a modulus of deformation than of elasticity when dealing with fill materials.

The value of E for a fill can be estimated from the stress strain curve obtained from triaxial testing. The value obtained is usually determined as the secant modulus between the origin and one third or one half of the peak stress, or over the actual stress range of most interest for the particular problem.

Typical values of E, for in tact rock in the Canadian shield is 70 GPa. A typical fill will have a stiffness value of 0.5 GPa. A high stiffness rockfill may attain a stiffness of 4 to 5 GPa. These values are for samples in triaxial testing. Fill is not stiff in comparison to intact rock.

2.4.3.1 Confinement

Fill in place will generally be subject to some confinement. This will help to increase its stiffness. Figure 2.7 illustrates the relationship. Curve one represents the hydrostatic loading case where the material is constrained by a uniform all round pressure. This implies that the fill has no shear strength, i.e. is a fluid, and it is unlikely that this stress situation would ever apply. Curve two represents the earth pressure at rest condition with one dimensional compression where lateral strain is not allowed. This is the situation with fill placed in narrow deep veins where loading is essentially one dimensional due to the convergence of the stope walls. Curve three represents the conditions typical of triaxial testing with partial constraint maintained with a constant confining pressure. Most practical fill situations will lie between curves two and three (Thomas et. al., 1979).

These curves are important because they indicate the importance of confinement to the ability of a fill to provide regional support and convergence control. The degree of confinement is equally important to a fill expected to perform as regional support as the internal properties of the fill itself. An important point to note that the degree of stiffness of a fill is a function of the amount of convergence which has taken place. At low convergence the stiffness presented by a fill is low, as convergence increases the stiffness increases. This can be viewed in terms of activating passive earth pressures through compression of the fill.

2.4.3.2 Binder

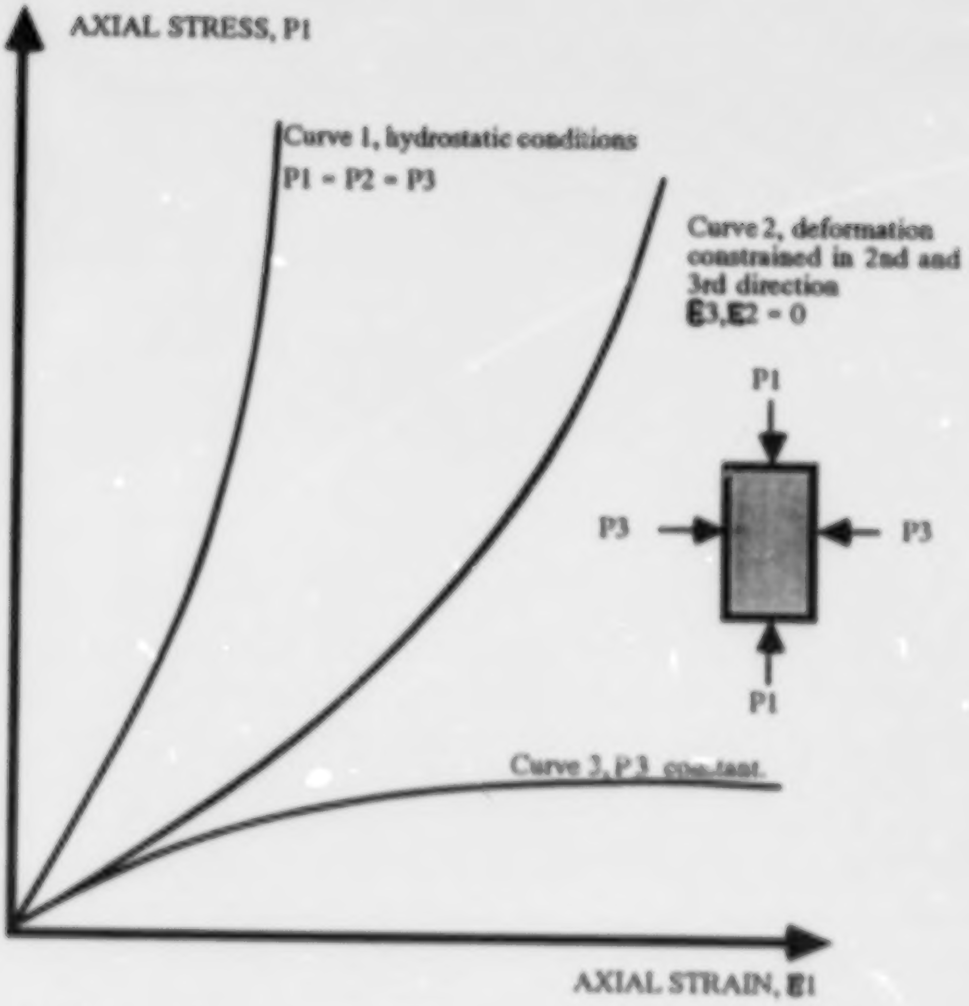
Addition of binder will increase the stiffness of a fill that is poorly confined. This was well illustrated by Swan, 1985 who proposed an empirical relationship between deformation modulus, E and unconfined compressive strength based upon assessment of 30 mine backfills and concretes. Figure 2.8 illustrates the relationship.

$$E = 0.21 \ UCS^{1.44}$$

where: E = deformation modulus, GPa
UCS = unconfined compressive strength, MPa

The addition of binder gives the fill unconfined compressive strength, so figure 2.8 can be considered an indication of the relationship between binder and deformation modulus for unconfined fill.

Confinement of backfill adds a overlying complication to the role of binder in improving backfill stiffness. It is recognized that binder may not improve the eventual stiffness which a backfill may exhibit, however it is suggested that a binder will decrease the amount of compression required to activate stiffness in a well confined fill. The action of binder on confined fill stiffness is illustrated in figure 2.9. This binder effect is important to fill placed as a means of convergence control.



Note: E=strain, P=stress

Figure 2.7, Effect of confinement on backfill stiffness.
(after Hendron, 1963, also shown in Thomas et al. 1979)

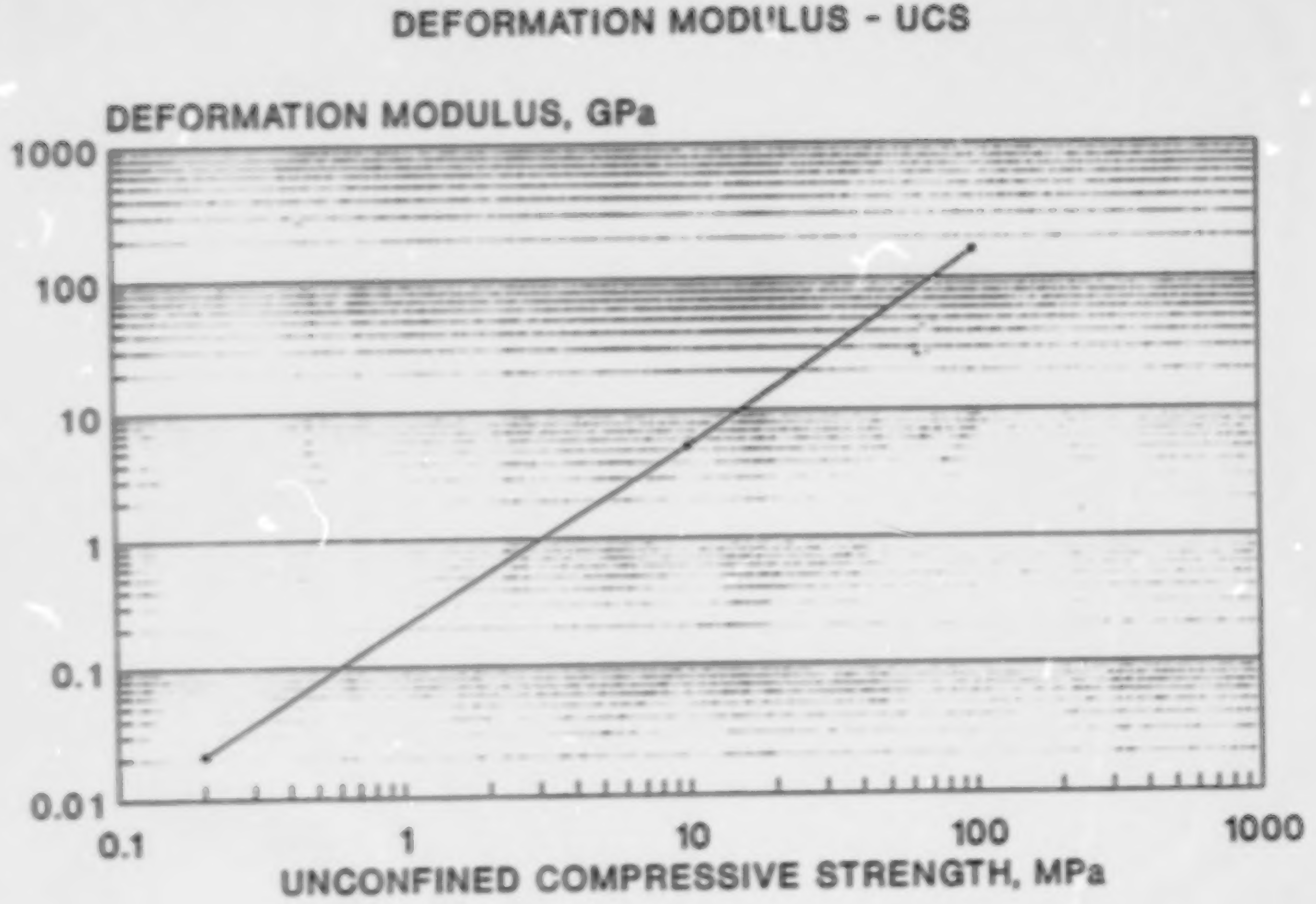
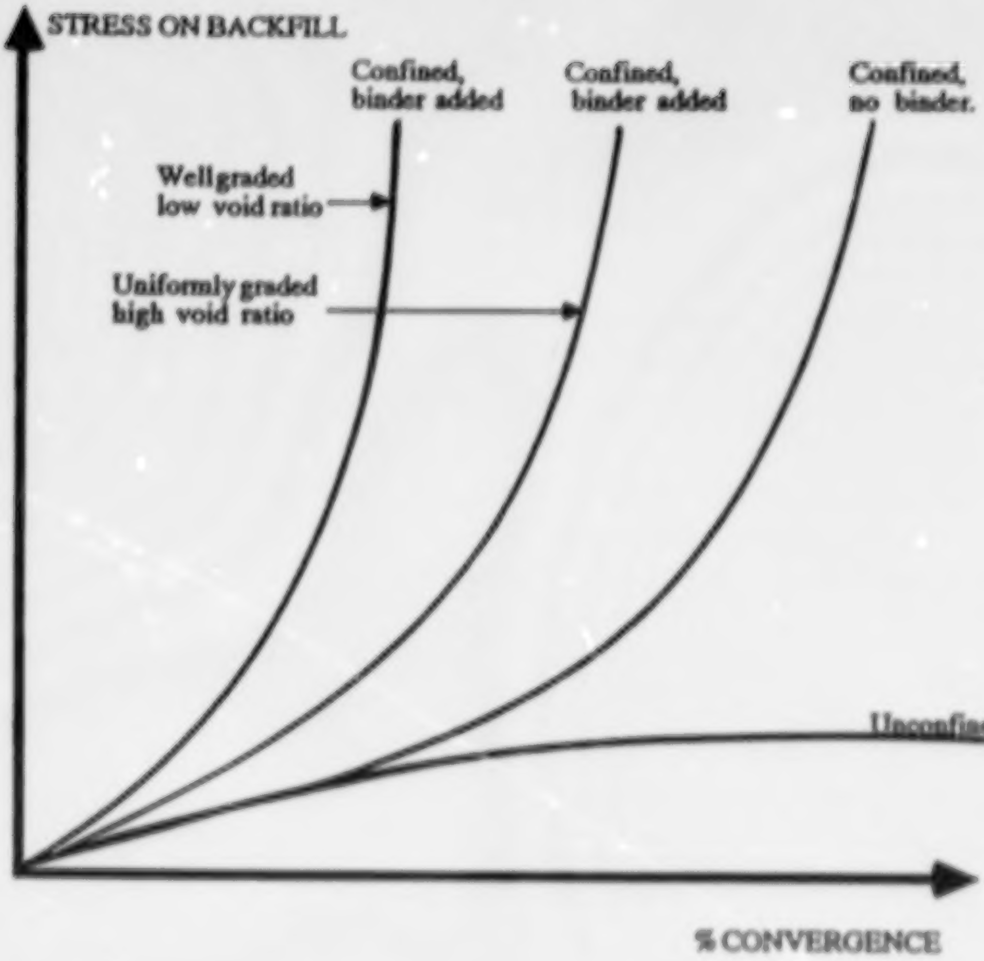


Figure 2.8 Effect of binder addition on stiffness of unconfined backfill, (after Swan, 1985)



It is suggested that binder decreases the amount of convergence required to activate the backfill stiffness response. The size distribution of the fill particles is also important.

Figure 2.9, Effect of binder on the stiffness of confined backfill.

2.4.4 Tensile Strength

The tensile strength of fill is important to its ability to resist damage due to blast vibration and to resist failure due to undercutting. Tensile strength of a fill will be limited and primarily a function of binder content.

Resistance to blast vibration damage is a function of dynamic tensile strength, which can be approximated as 5% to 15% of unconfined compressive strength. Arioglu, 1983, reported tensile strength of cemented aggregate fill at approximately 15% of UCS. Smith and Wong, 1982, reported tensile strength of cemented tailings fill to be approximately equal to 12% of UCS.

Measurement of tensile strength in backfill is difficult and so the factors affecting it are not well understood. It is known from work with rockfill at Kidd Creek that minimizing the voids in consolidated fill helps to improve resistance to blast vibration, see chapter 5.

2.4.5 Bearing Capacity

Bearing capacity is important to fill which is required to support mining activities on its surface. The bearing capacity of a fill is controlled by its shear strength, failure being by shear of a wedge of fill. Terzaghi's theory of soil bearing capacity is applicable to backfill. The following equation, based on the Terzaghi theory, assumes square footings (e.g. the footprint of mobile equipment tyres) acting at the surface of a cohesive soil.

$$Q = (1.3CN_c) + (0.4wBN_w)$$

where:

Q = bearing capacity, kPa
C = cohesion of the soil, kPa
w = unit weight of the soil, kPa/m^3
B = width of bearing contact surface, m
N_c = dimensionless bearing capacity factor due cohesion
N_w = dimensionless bearing capacity factor due unit weight

The factors N_c and N_w can be obtained from soil mechanics text books, figure 2.10 is a copy. It is obvious from the equation that cohesion plays a predominate role in determining bearing capacity.

The bearing capacity of unconsolidated hydraulic fill is controlled by its small apparent cohesion. Apparent cohesion is a result of surface tension in the water partially filling the voids in the fill. The magnitude of apparent cohesion is a function of the moisture content, for which there will be some optimum value. A fill which is fully saturated or completely dry will exhibit no apparent cohesion. The bearing capacity of a well graded unconsolidated hydraulic fill is generally sufficient to support mining activity.

The bearing capacity of unconsolidated rockfill is a function of its high friction angle, especially after compaction. Rockfill particle size is generally too great for any apparent cohesion to develop.

Addition of a binder may be considered to improve the bearing capacity of a fill. Cemented fill floors are often used to control dilution and to provide additional traction rather than to improve bearing capacity.

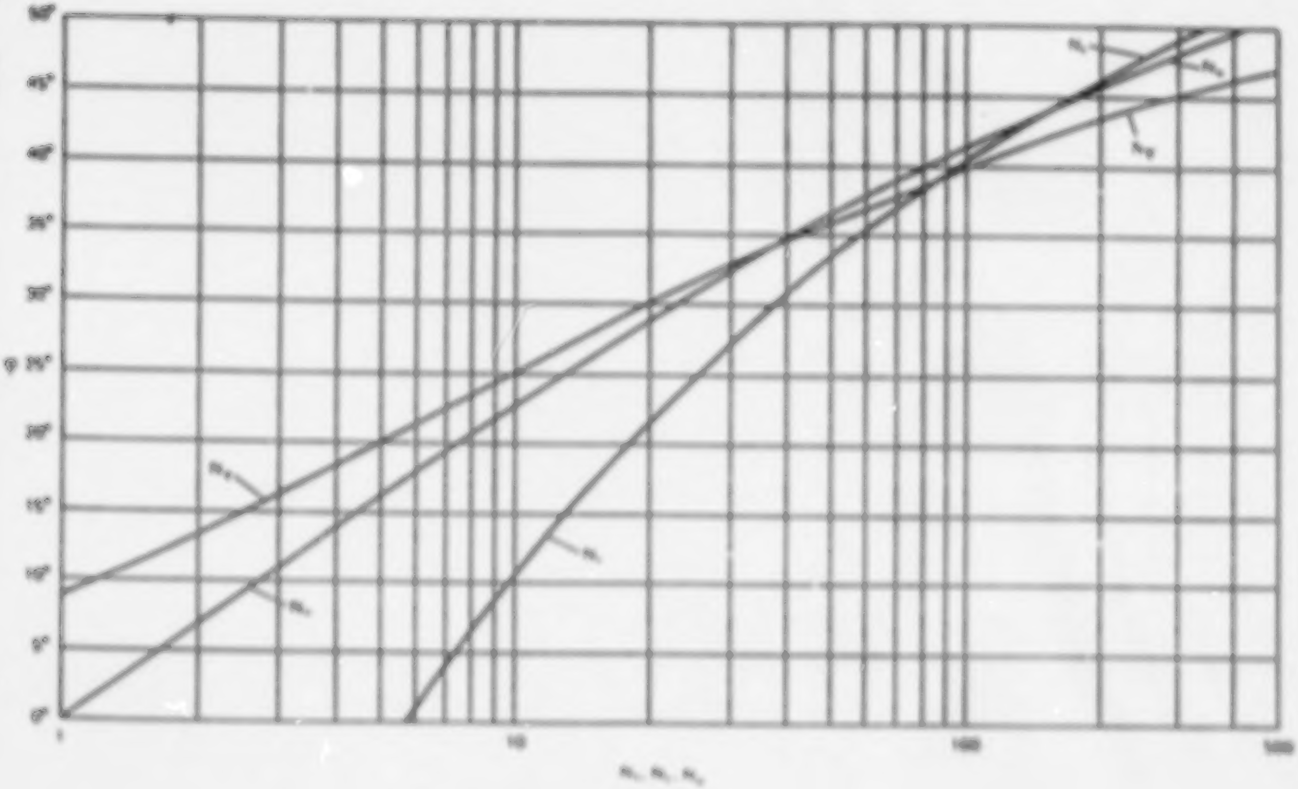


Figure 2.10 Bearing capacity factors for shallow footings, (after Craig, 1978)

2.5 BINDER ADDITION

Addition of a binder to backfill will add shear and compressive strength by providing true inter particle cohesion. The effectiveness of a binder is affected by both the material properties of the backfill and the conditions under which it is placed. The action of a binder is the result of a chemical reaction and so time plays an important role in the rate of strength development of consolidated backfills. The characteristics of consolidated backfill are analogous to concrete.

2.5.1 Consolidated Fill Strength

The mechanism of a binder performance in backfill may be described in two ways;

1. Sufficient binder must be added to fill the voids between fill particles to ensure good bonding and inter particle friction. The fewer the voids in the fill, the less binder is required. This is a function of the particle size distribution and the efficiency of particle packing. See figure 2.11.

2. Sufficient binder must be added to fully coat the surface of all the backfill particles to ensure good cohesion at the points of contact. This implies that fine backfill materials will require more binder than coarse materials.

In practice both mechanisms are applicable and act simultaneously, the importance of each varying with the aggregate size and size distribution. Hydraulic fills tend to be controlled more by void filling mechanism while in coarser rock fill the surface area mechanism tends to be predominant.

Previous discussion has indicated that the size distribution of a fill may influence the inter particle friction of unconsolidated fills. The binder mechanisms are considered to act in parallel to this strength control mechanism.

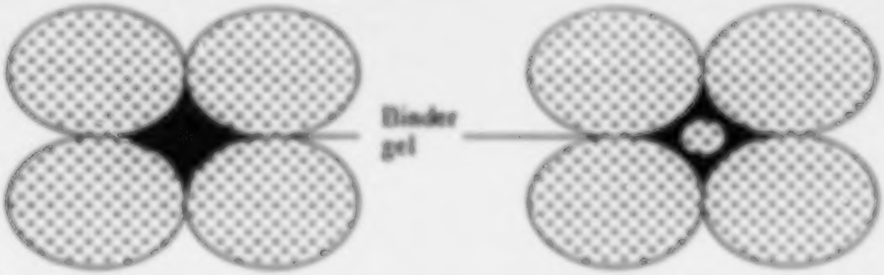
The relationship between strength of concrete and water binder ratio is well known; as the ratio increases the strength declines rapidly. The action of a binder is a chemical reaction and as such it is sensitive to the the chemical environment. An excess supply of water will cause "over hydration" of the binder, where the binder hydrates to produce a weaker cementing gel than when water supply is limited. Backfill, both hydraulic and rockfill, is often placed with water binder ratios much higher than those used in concrete applications. In hydraulic fill this is a consequence of the excess water required for transport. In rockfill it is a consequence of the mixing procedure or the density of the binder slurry. Hydraulic fill has a significantly worse ratio than rockfill, (rockfill is a more efficient user of binder than hydraulic fill). Common water binder ratios for concrete and the expected strengths are shown below in table 2.2. Compare these to the common backfill values of 10 for hydraulic fill and 1.0 for rockfill. Figure 2.12 is taken from a paper which discusses paste fill technology (Lerche, Renetzeder, 1984); it gives a good illustration of the influence of water cement ratio on the strength of a mine backfill.

Water : Binder	UCS, MPa
0.7	15
0.6	21
0.4	39

Table 2.1, Strength of concrete at various water binder ratios.
(After US Dept Interior, 1975)

In summary, the strength of a consolidated backfill may be considered as the function of the interaction between;

1. Water : Binder ratio
2. Absolute size and size distribution of the fill aggregate
3. Void ratio of the fill in place
4. Quantity of binder added



Situation with no grading control
- binder gel must fill all voids

Situation with good grading control
- small particles occupy the voids reducing the amount of binder gel required

Figure 2.11, Model demonstrating benefits of fine particles in hydraulic fill. (after Thomas et. al., 1979)

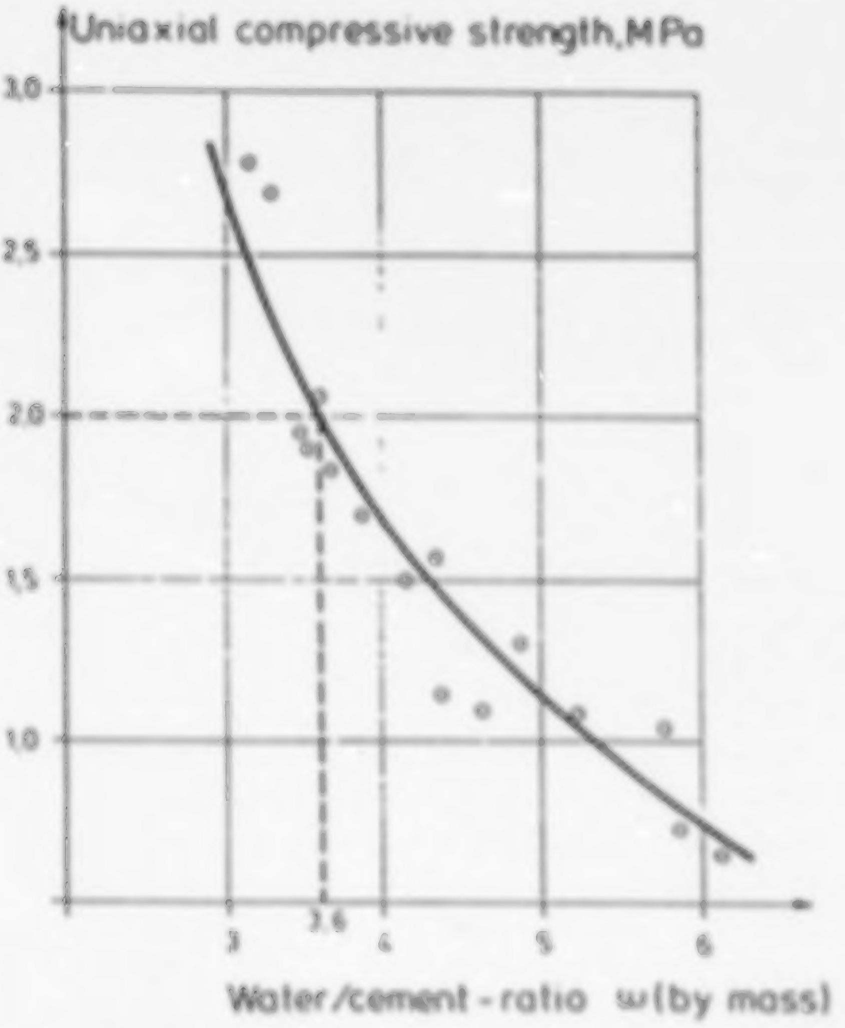


Figure 2.12 Effect of water cement ratio on backfill strength. (after [illegible], 1984)

2.5.2 Design Equations

A number of design equations have been proposed which attempt to predict the strength of backfill materials. These equations use the factors listed above in various forms in combination with empirical or experimental constants.

2.5.2.1 Talbot Curve

In the concrete industry the Talbot curve is often used to determine the required cement addition to achieve a desired concrete strength. The Talbot curve is defined by:

$$P_{(u)} = 100(u/u_m)^n$$

where: $P_{(u)}$ = percent passing by weight for sieve size u and for particle size u_m

n = experimental constant.

Experiments by Talbot and Richart (1923) showed that a specific value for exponent "n" would minimize void ratio of a concrete mix, thereby maximizing concrete strength. A Talbot-based design procedure can be employed to design backfill mixes comprised of aggregate material bound by a matrix consisting of fine aggregate and cement or other binder. Shape effects of the particles are neglected as this theory is based on an assumption that all aggregate particles are spherical. It is also assumed that the size of the aggregate is variable and that it is arranged randomly.

2.5.2.2 Hydraulic Fill Equations

Swan (1985) has proposed a design method for backfill materials based originally on the Talbot curve approach. The two most important theoretical parameters which are used for this design method are d_{avg} which is the mean free distance between aggregate particles in the mix, and s_p, the aggregate's specific surface area for a constant water cement ratio. Knowing the fill aggregate particle size distribution, its porosity and volumetric cement content allows the calculation of s_p and d_{avg}. This is done with a series of equations detailed in Swan's paper. Swan developed a dimensionless "binder number" which related the volume of cement added, C_v, to the product of d_{avg} and d_p, i.e. "binder number" = $[(C_v/(d_{avg}s_p))]$. The binder number was plotted against unconfined compressive strength for 68 mine backfills. An empirical relationship was developed as follows;

$$UCS = 0.283 \ (C_v / d_{avg} \ a_p)^{2.36}$$

where; C_v = volume of cement in a unit dry volume of freshly placed backfill, wgt %

d_{avg} = mean free distance between aggregate particles in the mix

a_p = aggregate's surface area for a constant water cement ratio

Swan's equation is equally applicable to rockfill as to hydraulic fills.

Mitchell and Wong (1982) have proposed a design equation which relates the porosity, water cement ratio and binder content to unconfined compressive strength of hydraulic fill. This equations considers porosity and water cement ratio as the principal determinant of strength. It is based upon plotting the function $w^{-a} n^{-6}$ against uniaxial strength to form strength lines at different binder addition rates. The slope and intercepts of these lines are expressed as experimental constants. Laboratory testing is used to determine the constants K_1, K_2, and **a** in order to predict the effect of variations in binder and water on fill strength. Mitchell and Wong state that data from Weaver and Luka (1970) indicate that a can vary from 1.1 to 1.7 for tailings fill. The design equation can be expressed as;

$$UCS = K_1 \ c^a \ (n^{-6} w^{-a} + K_2)$$

where; UCS = unconfined compressive strength, kPa

n = porosity

w = water content (%)

c = binder content (%)

a = experimental constant

K_1, K_2 = constants to be determined experimentally

Chen and Jiao (1991) have reported an empirical design equation for hydraulic fill based upon the observation that compressive strength depends upon the ratio of cement to tailing and the slurry pulp density. The statistically derived equation is;

$$UCS = Ae^{BC} + N(C_g - 0.65)$$

where;

UCS = uniaxial compressive strength, MPa

C = ratio of cement to tailings, %

C_g = solids concentration of fill slurry placed underground, by weight 65 to 85%

N = constant, 2.5 for consolidated fill

A,B = constant depending on the solid grain size and its slurry density;
for tailings with slurry density of 65% solids by weight,
A = 0.2359, B = 14.494
for < 2.5 mm sandfill with slurry density of 65% solids by weight
A = 0.257, B = 19.628
for <1.2mm sandfill with slurry density of 65% solids by weight
A = 0.255, B = 16.459

2.5.2.3 Rockfill Equations

Kidd Creek Mines Limited have developed and empirical relationship between compressive strength and cement content for their consolidated rockfill;

$$UCS = 1.5e^{0.25C} \qquad \text{for } 2 <= C <= 10$$

where;

UCS = uniaxial compressive strength of rockfill, MPa

C = portland cement content by weight % of -4cm aggregate

Ariogla (1984) has developed a design equation for rockfill based upon statistical assessment of experimentalvalues;

$$UCS = A.(W:C)^{n}$$

where;

UCS = unconfined compressive strength, MPa

W:C = water : binder ratio by weight

A,n = constants for the particular mix of rock
A = 80.268, n = -1.536 for the rockfill investigated by Ariogla.

Swan's design equation based upon the binder number can also be applied to rockfill design.

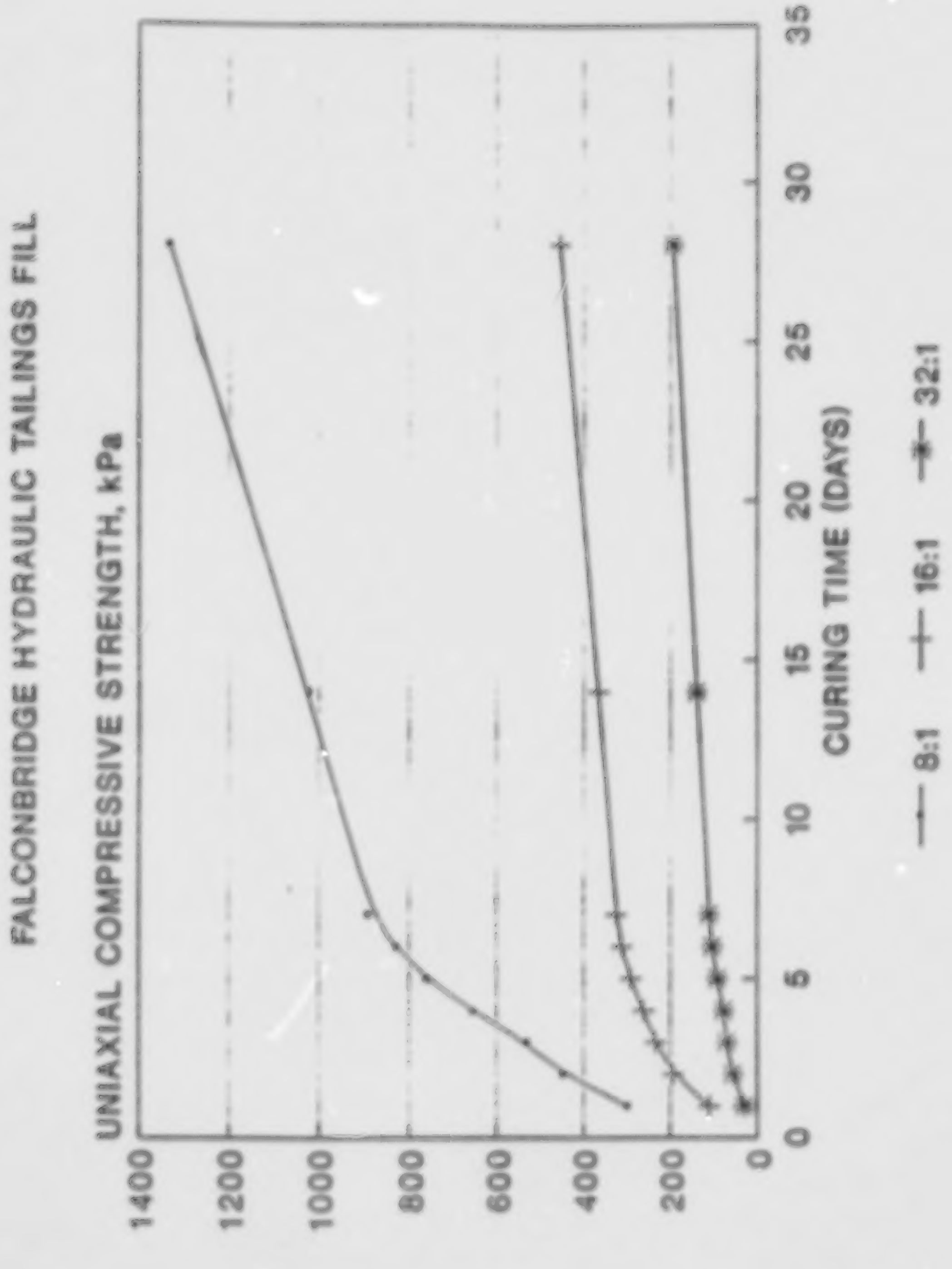


Figure 2.15 Characteristic strength development with time for Portland cement consolidated fill

2.5.3 Binder and Time

Binders develop strength slowly. Figure 2.15 shows the characteristic curve showing strength development over time for a typical hydraulic tailings fill with portland cement. Different binders will exhibit different strength development rates depending upon their chemical and physical make up, and how they react with the fill material. The rate of strength development is a major criteria in the selection of a suitable backfill binder. Strength development rates interact heavily with the mining schedule. In general the more binder added the faster will be the rate of strength development.

2.5.4 Curing Temperature

The temperature at which a consolidated backfill is placed and cured is a significant variable in determining the rate of strength development and the ultimate strength achieved. In general, higher curing temperatures will result in faster strength development. There is an optimum temperature (around 38° C as reported by Thomas et. al. 1979) after which higher temperatures result in lower ultimate strength due the formation of an inferior quality cement gel. The difference in rate of strength development may be quite dramatic, strength increased by a factor of two at 28 days curing between temperatures at 5°C and 40°C. Any laboratory determinations of fill strength should be conducted at curing temperatures representative of what is actually encountered underground. The actual curing temperature is not a variable which the backfill engineer has much control over in designing backfill.

2.5.5 Chemical Composition

The chemical and mineralogical composition of the fill material may influence the performance of a binder. This is especially true for hydraulic tailings fill which may contain a significant quantity of reactive sulfide minerals. These sulfides have the potential to oxidize which may cause expansion cracking and loosening of the binder cohesion over time and the acidic sulfides may attack the binder hydration products. The sulfide corrosion of concrete is well documented. The oxidation problem is probably not as great as might be expected because fill underground is usually at least partially saturated and is comparatively thick, so oxidation is limited in practice to the surface of exposed fill. The potential for problems should be carefully considered if reactive sulfides are to be used as backfill.

There is some potential for positive interaction between the binder and the chemical composition of fine hydraulic fill materials. The chemistry is not understood, but the performance of ferrous slag as a binder with Sudbury area tailings fill was shown during this project and by other investigators to be unusually good compared to expectations from normal concrete technology.

The chemical composition of rockfill materials is not of as much concern. The surface area for any reactions to take place is small and if any occurs is likely to be extremely slow.

2.6 CONDITIONS OF PLACEMENT

The conditions of placement will influence the mass properties and hence the strength of a backfill. By conditions of placement we refer to factors such as the pulp density of hydraulic fills, the method of mixing rockfill with its binder, the orientation of the pour point relative to the geometry of the void being filled, compaction, efficiency of the drainage supplied for hydraulic fills, degree of confinement, and other local factors. The degree of particle segregation and in place density are the properties most affected by placement conditions and of most concern in designing a backfill.

They way in which conditions of placement affect backfill are discussed in detail for rockfill and hydraulic fill in the relevant chapters which follow.

2.7 THEORETICAL BACKFILL DESIGN

2.7.1 Failure Modes of Backfill

Several modes of backfill failure have been identified.

1. Failure due to insufficient stiffness, leading to excessive deformation under load.

2. Failure due to saturated fill which is improperly drained and can liquify under dynamic loading.

3. Failure by sloughing.

4. Failure due to inadequate bearing capacity under surface loading from mobile equipment, weight of ore piles and human activity.

5. Failure due to insufficient shear strength resulting in slabbing or wedge failure of exposed vertical fill faces.

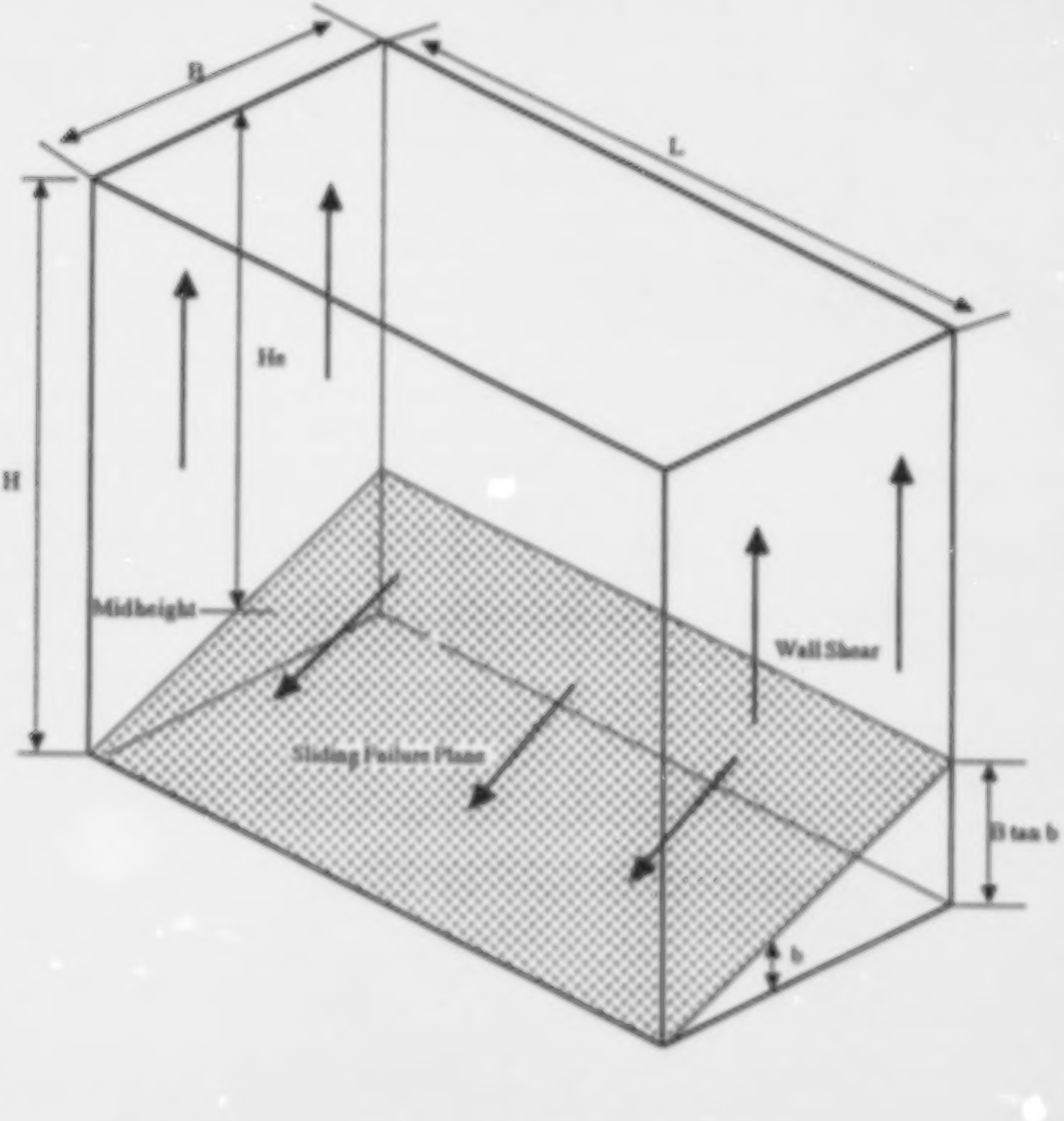

B
L
He
H
Midheight
Wall Shear
Sliding Failure Plane
B tan b
b

Figure 2.14 Shear failure mode diagram, (After Mitchell and Smith, 1979)

The type of fill failure that is most often discussed is the failure of a free-standing face of fill into a void created by the mining of an adjacent pillar. Reports published in the public domain literature and experience have shown that typical failures in high, exposed backfill faces originate as a series of closely spaced vertical fractures or discontinuities in fill caused either by shrinkage during curing or by vertical drainage channels during percolation of the fill. These run parallel to the backfill walls. The proposed failure mode is shear at the base of the fill intersecting these vertical discontinuities causing a wedge to fall out.

2.7.2 Stability of a Free-Standing Face of Fill

The simplest way to analyse the stability of a free standing face of fill is to assume a wedge type failure mode controlled by shear strength. Figure 2.14 shows this mode of failure. It is possible to analyse such failures using a limiting equilibrium approach. For the wall to remain stable, the resisting forces in the failure plane must exceed the driving forces. This is expressed as a factor of safety, *F.S.* , where

F.S. = (Resisting forces in the failure plane) / (Driving forces in the failure plane)

For the fill wall to be stable, F.S. must be greater than 1.0.

To determine the resisting and driving forces, the following procedure is used. The driving force is a function of the failure angle and the force parallel to the failure plane as a result of the net weight of the block of fill, which is equal to the block weight less the wall shear component. The net weight is given as;

$$W_n = L\ B\ H_e\ w - 2\ C\ B\ H_e$$

where:
W_n = net weight of block of fill, kg
L = strike length of exposed fill, m
B = width of block of fill, m
H_e = effective height of block of fill, m
w = unit weight of fill material, kN/m^3
C = cohesion of fill material, kPa

The actual driving force along the failure plane is then calculated as;

$$F_D = W_n \sin \beta = (L\ B\ H_e\ w - 2\ C\ B\ H_e) \sin \beta$$

where:
F_D = driving forces along the plane of failure, kN
β = angle of plane of failure, where $\beta = 45 + \varnothing/2$
$\varnothing$ = angle of internal friction

The resisting forces are a function of cohesive and friction forces along the sliding plane,

where: $C_F = C\ L\ B / \cos \beta$ = cohesive force
and $F_F = W_n \cos \beta \tan \varnothing$ = friction force

The factor of safety can now be re-defined as

$$F.S. = (C\ L\ B / \cos \beta + W_n \cos \beta \tan \varnothing) \Big/ ((L\ B\ H_e\ w - 2\ C\ B\ H_e) \sin \beta)$$

It should be noted that properties C, w and Ø are determined in the laboratory.

The maximum backfill face height that can be exposed for a given stope width can now be calculated by substituting 1.0 for F.S. assuming that $H_e = H$, the stope height and solving the equation.

$$2C = wH/(H/L + \tan \beta)$$

To calculate the critical height of a exposed fill face Mitchell et al (1982) assumed the conservative condition of friction angle, Ø, equal to zero.

If $\varnothing = 0$
Then $\tan \beta = 1$
And assume $2C$ = Unconfined Compressive Strength, UCS, kPa

Hence $UCS = wH/(H/L + 1)$

This equation provides a relationship between fill compressive strength and stope geometry which must be met in order to avoid shear failure of the fill pillar. This relationship can be used to determine cement requirements for backfilling a stope against which pillar mining is to occur. Because it is easy to measure most backfill strengths are quoted in terms of unconfined compression, UCS, hence having an equation which relates UCS to stable height is very useful, even if it does require a few assumptions.

1. **Wall cohesion equivalent to fill cohesion.** Stope walls are left rough due to the affects of blasting. This roughness would lead one to expect that the shear resistance offered by the fill rock contact would be higher than the shear resistance provided by the fill alone. If this is the case then we would expect shear failure to occur in the fill close to the wall rather than through the rock fill interface. This assumption is good and not overly conservative.

2. **Effective height equal to the fill exposure height.** In the usual case were $B << H$ this is a good approximation. If $B << H$ is not true, then the assumption becomes more conservative. The following equation could be used.

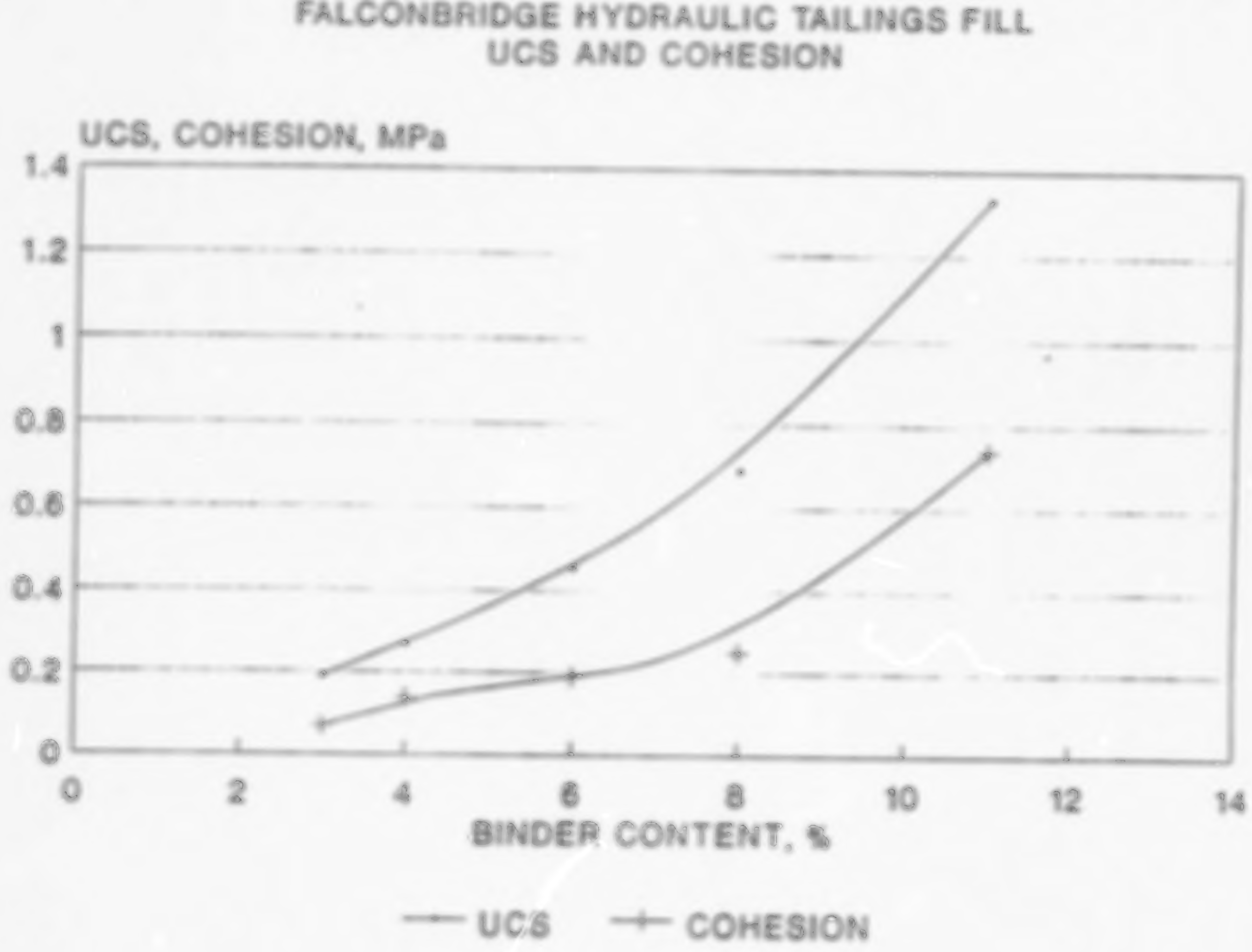
$$UCS = w\ (\ 2H - B\ )/[((\ 2H - B\ )/\ L) + 2]$$

This equation gives lower UCS values, but it should be used with caution because as the width of a pillar increases, the potential for failure modes other than the confined block failure mode increases.

3. **Angle of internal friction of fill equal to zero.** This means that the fill is being treated as a frictionless material. By assuming this a conservative estimate of the required UCS is obtained. The assumption of UCS = 2 C is inherent to the $\emptyset = 0$ condition. Research work done by the Canada Cement Company and by Queens University on Falconbridge tailings fill shows that the assumption is valid, if not overly conservative, in the 0 to 11% cement content range. See figure 2.15.

2.7.2.1 Effect of fill arching

The above analysis of fill failure based upon a confined block model does not consider the affect of fill arching which is known to occur and assist fill stability. Research by Askew et. al. 1978 concluded that fill arching will reduce vertical pressure at the base of a stope due to the weight of soil above to about 30% of the value calculated assuming no arching. Thus the value of the driving force used in the above calculations of stability could conceivably be reduced by 70% for a more realistic estimate of fill stability. However it must be remembered that backfill is not a homogeneous material and that discontinuities will reduce the fill strength from that measured in the laboratory. So that it is probably fair to disregard the affect of fill arching in designing for fill stability.



PORTLAND CEMENT AS BINDER

Figure 2.15 Uniaxial compressive strength and cohesion for Falconbridge tailings fill.

2.7.2.2 Assumptions

It should be remembered that the design approach presented above is rather crude and conservative. The assumption of shear wedge failure being the critical failure mode is also a rather gross simplification of a complex system. None the less the approach is still valid as a first pass design tool and in the absence of any more sophisticated modelling techniques. Practical experience with the stability of actual faces will be the operators best guide to strength requirements, because it will reflect the true failure mechanism which may not be shear wedge related.

The assumption of a vertical face of fill being exposed by pillar extraction is another simplification which should be considered with care. In large blast hole stopes at depth the completed stope profile will rarely be the vertical walled shape it was designed to be. Overbreak due to blasting and spalling due to wall convergence will often result in a convex wall profile. When the adjacent pillar is mined the fill in the original stope will have a convex face exposed. It is difficult to place a fill strong enough that the convex part of the face does not fall off to bring the face back to a vertical one.

2.7.3 Subsidence and Convergence Control

Backfill is frequently placed to control subsidence at the surface or the convergence of ground into mined voids. This is a regional support role. The mechanisms of how backfill fulfills this role is matter of some research and detailed discussion is outside the scope of this report, however a brief review is presented.

The resistance to convergence or subsidence by a backfill may be considered in terms of passive earth pressure. Some convergence must take place in order to activate the resisting forces. The amount of convergence required to activate these forces is a matter for some discussion, but it is fair to say that the stiffness and confinement of a fill are important parameters. Addition of a binder seems to reduce the amount of convergence required to activate the passive earth pressures, although the eventual magnitude may not increase because of the binder. In addition the efficiency of coupling between the stope walls and the fill is critical. Good coupling in shallow dipping ore bodies is often difficult to achieve. Timing of backfill operations is therefore important in the quality of support provided by the fill because if the delay between mining and backfilling is too long, much of the convergence of the surrounding rock will have already occurred. This will result in a loss of the fill support capacity (Hedley 1984).

The ability of unconsolidated fills, placed in confinement , to provide regional ground stability is dependent upon the friction angle of the material. Cohesion still plays a part, but friction angle is by far the more important strength component when fill is in confinement. The resistance to movement of rock into confined fill may be considered as passive soil pressure. The magnitude of this passive earth pressure is given by Rankine's theory of earth pressure;

$$P_p = K_A w z + 2C\sqrt{K_A}$$

Where:
P_p = Rankine passive earth pressure, kPa
K_A = $(1 + \sin\phi)/(1 - \sin\phi)$
ϕ = friction angle, degrees
C = cohesion, kPa
w = unit weight, kN/m^3
z = vertical depth from soil surface, m

The passive earth pressure is only activated by movement of the rock into the fill. The amount of movement required to activate the earth pressure is dependent upon the denseness of the fill, wall friction, action of binder addition, and other factors not well understood. Not enough is known about the development of stresses in fill and how rock transfers load to a fill to be able to use the equation of passive earth pressure with any confidence, however it does indicate that fill with high friction angle will perform better than one with a low friction angle. Hence effort is made to place low porosity well graded fills.

CANMET has published a paper in which a good example is given of calculations to determine displacements and convergence around a stope (Hedley 1984). This analysis helps to determine the degree to which a backfilled stope is compressed (i.e. becomes an active rather than passive system). The stope is considered as a thin slot with rigid abutments. The convergence distribution across the slot is;

$$S = [(4 \ (1-\nu^2)S_o) / E \ ] \ [(1 + cx/2) \ \sqrt{(a^2 - x^2)}]$$

where:
S = convergence
ν = Poisson's ratio (typical value for hardrock mines in the Canadian Shield = 0.2)
a = half span
x = distance from centre of span ($-a \leq x \leq a$)
c = Sin θ / H, where θ = dip angle and H = depth below surface
S_o = virgin stress perpendicular to orebody (see further below)
E = modulus of elasticity
(typical value for hardrock mines in the Canadian Shield = 70 GPa)

It should be noted that at depths greater than 1000 metres, cx/2 becomes insignificant unless the stope span is very large.

If the principal stresses are assumed to be vertical and horizontal, then the perpendicular virgin stress can be calculated as;

$$S_n = (w\ H\ /\ 2)\ [(1 + k) + (1 - k)\ \mathrm{Cos}\ 2\beta]$$

where: w = unit weight of rock mass
(typical value for hardrock mines in the Canadian Shield = 0.027 MN/m^3)
k = ratio of horizontal to vertical virgin stress
(typical value for hardrock mines in the Canadian Shield = 2.0)

The maximum convergence in backfilled stopes occurs near the center of the span and is calculated as;

$$\delta_{max} = (4\ (1-\nu^2)\ S_n\ a)\ /\ E$$

where: δ_{max} = maximum convergence
ν, S_n, a = as in previous equations
E = modulus of elasticity
(typical value for hardrock mines in the Canadian Shield = 70 GPa)

2.8 DETERMINING THE IMPORTANT FILL PROPERTIES

Having identified the properties of backfill which are important in determining how it will perform in the role required of it, one must be able to measure these properties. Original measurement of the important properties for design purposes requires a series of laboratory tests. Once the backfill is being placed it is a good idea to check the in place values with the laboratory values, remembering that segregation will create variable quality backfill throughout the fill mass. Appendix H of this report details some procedures for determination of the backfill properties both in the laboratory and in place. The text book, Fill Technology in Underground Metalliferous Mines by Thomas et. al. 1979, also gives an excellent revue of procedures for determination of backfill properties.

The laboratory tests required would include;

1. Compression testing, both uniaxial and triaxial

2. Direct shear box testing

3. Percolation rate testing

4. Density analysis (solids, bulk)

5. Sieve size analysis

6. Consolidation or compressibility

7. Moisture content and liquid limit determination

A number of methods for determining in place properties of backfill are also possible;

1. Deformation modulus measurement with a pressuremeter from which the modulus of elasticity may be determined.

2. Plate bearing test for determination of bearing capacity from which cohesion and friction angle may be determined.

3. Direct shear test

4. Uniaxial compression tests.

In place testing of backfill is not easy. For representative results the testing should be done away from the edges of the filled void where segregation effects are often the most extreme. This implies excavation into the backfill mass, which implies disturbance of the backfill and hence backfill in a non representative state. Many of the instruments used for in place determination of backfill properties were developed for use in rock which is much less prone to disturbance. The procedures for in place determination of backfill properties are not well developed and much more work is required to ensure their reliability.

2.8.1 Sampling Backfill

Coring and/or drilling in backfill is a highly problematic activity for many reasons. First, the use of water or other drilling fluids required to cool coring bits is detrimental to the sample which can undergo severe changes in physical and chemical properties as a result of contact with the drilling fluid. Also, backfill materials can swell during collection when they are being removed from the coring machine. There can also be considerable breaking of samples during extraction due to the rotation and pulling motion of the drill. This is especially evident when there is segregation of coarser particles and the coring has intersected a band of weakly cemented fill. Consequently, samples obtained by conventional diamond drill coring techniques are relatively disturbed and hence, non representative of the true conditions in situ.

Some coring and drilling methods have been proposed to offset the effects mentioned above. For example, hollow tube rotary drilling using compressed air has been suggested as a possible alternative to using water as a cooling fluid. Also, bagging of the samples immediately when they are retrieved is recommended so that the in-situ moisture conditions are preserved.

Falconbridge Limited has researched and developed a technique for coring in fill which was found to be highly successful. The technique is described in a contract report to CANMET (Falconbridge Ltd. 1988). Core recoveries using the Falconbridge technique were as high as 95 percent, even on very weakly cemented backfill. The technique employed a Diamec 251 diamond drill which was modified using a low rpm and high torque unit with a split inner tube core barrel. The flushing agent used for drilling consisted of a granular co-polymer of sodium acrylamides and acrylates. This substance was found to have minimal effect on the moisture content of the samples retrieved. Although the procedure was found to be technically successful, it was not economically attractive as the equipment and modifications were costly. The equipment also required a highly qualified operator, making for high operational costs.

The location of backfill cores taken for testing should be carefully considered. For example, when mining under a sill mat, one ten-foot long cored fill sample should be taken for every 50 foot length of mining advance. Such samples should be subjected to strength tests and to cement content analysis. In other cases, in-situ cored samples should be taken periodically. When binder contents are questionable due to visual indications of segregation or excessive drainage, samples should be cored at various lateral distances away from the pour points to establish the binder depletion pattern.

2.9 MODELLING

2.9.1 Physical Modelling

Various forms of physical modelling can be used to simulate the behavior of backfilled stopes on a small scale in the laboratory. Although physical modelling of a complex mining system such as that involving backfill cannot be considered as a general design tool, it is very important in determining possible modes of response under controlled conditions (Aitchison 1973). Therefore, physical modelling can be used as a research tool and as a precursor to numerical modelling because it can yield values which would be used as input into computer models.

There are three main types of physical models. These are small ("bench") scale models, large scale models and centrifuge models. Bench scale models can be used to predict particle or cement segregation effects, banding of fills in horizontal or vertical stope filling zones, induced bulkhead pressure conditions under varying drainage conditions and free-standing height of exposed fill faces. Bench scale tests can also be used to help determine the best transport method for the fill by testing the transportability of the backfill under different modes such as pneumatic, hydraulic, gravity or pumped transport.

Bench scale modelling is usually done by scaling down a stope to a timber box measuring around 1 to 2 meters in its largest dimension. Some variations on the timber box do exist, though, such as in the case of Mitchell & Roettger (1984) who have used a cylindrical clear plastic bench model stope to demonstrate the effects of fill drainage and fill pressure on bulkheads.

A good example of bench scale modelling to determine the strength requirements for a stable vertical face of fill is given in Mitchell et al (1982). Here, the model fill was poured in wood forms consisting of 50 mm thick plywood supported by steel and wood bracing along the back and horizontally placed timbers along the front. The front timbers could be removed gradually to simulate the incremental exposure of the free face of backfill as mining progressed. The forms were 2 metres high by 1.8 metres wide by 0.9 metres deep. The authors warned that when using physical models in the laboratory, temperature variations in the building can affect curing rates and therefore a temperature controlled environment should be created to surround the model.

Small scale physical models are not considered to be extremely reliable because they cannot offer true physical representations of in-situ conditions. Also, size effects and scaling factors must be known in order to assess the effect of down sizing on the true conditions of the stope. And because of the small quantity of backfill required for bench scale models, it is usually not practical to prepare a batch from an actual fill plant and therefore, batches for such models are usually mixed by hand in the laboratory. Hence, the backfill mix will not be fully representative of the actual one used in the stope being modelled, thereby adding further potential error to the modelling exercise (Archibald 1989).

Large scale models are those that are created at scale factors of less than 20:1 (Archibald 1989). These are generally more representative than bench scale models and because of their size, they justify batching of the backfill mix in the actual fill plant. Such models can also incorporate the effect of the transport method on the backfill being poured and that of the curing environment if they are built underground. They have a further advantage of avoiding the scaling effects that are a problem in bench scale tests. Their only drawback is that they are very costly to build and maintain and they have limited application.

Centrifuge models are scale models which are tested in a centrifuge machine such as that which was employed by Falconbridge in its subcontract to Queen's University (see Chapter 6 for examples of centrifuge testing of novel types of backfill). Such tests were conducted to determine the response of layered fills, geogrid reinforced fills and sloped fills to mining stresses.

Centrifuge modelling uses geometrically shaped models at a scale of approximately 100:1. This type of modelling helps to predict the deformation of large in-situ fill masses by the acceleration of the models in the centrifuge. It is believed that the stresses producing responses such as deformation and failure which are comparable to the actual in-situ fill mass can be duplicated on scale in the centrifuge. There is a great deal of published information to which the reader can refer for details on centrifuge modelling of backfill, such as Mitchell 1988a, Mitchell 1988b, Mitchell 1986, Mitchell & Smith 1987 or Mitchell 1989.

In addition to the three main types of physical models described above, there are also other model types which are more unusual, but which are created to simulate a desired effect in backfill. One example is the drop box used to determine the relative resistance of various types of backfill to impact damage (Mitchell & Stone 1987). Here, samples consisting of fills layered with strong and weak cement and some containing fibre reinforcement elements were subjected to various drop heights to determine failure planes and relative resistance to impact. Such models are acceptable in providing relative qualitative rather than quantitative data for one type of backfill in comparison with another.

2.9.2 Numerical Modelling

According to Chen et al (1983), finite element modelling was first applied to backfill in the mid 1970's by Pariseau et al (1973, 1976). Early attempts at employing FEM for backfill helped develop and strengthen many concepts regarding fill that were still unclear at the time. For example, it was found using finite element analysis that the support capability of fill increases with the stiffness ratio, a concept which proved to be true in the field.

The application of finite element modelling (FEM) in helping to determine the stresses and deformations produced in a rock body or in backfill as a result of mining operations is now widespread. Using computer modelling techniques, one can rapidly "foresee" the effects of different layout geometries on the stress conditions in neighboring areas. Modelling can also be used to determine the influence of the pre-mining stress field and the depth of the mining operation on backfill performance and to predict the influence of mining geometries on roof and pillar stability, ore losses, dilution and backfilling costs. Modelling is also helpful in predicting stresses in backfill and the stress redistribution in the wall rock and fill when pillars are being mined adjacent to previously filled stopes. Also, the response of a whole system to changes in any one parameter (i.e. sensitivity studies) can be predicted using computer modelling. And finally, the economics of a backfill system can be optimized by using numerical modelling with sensitivity analyses.

Two-dimensional numerical models are much simpler to operate and require less CPU time to run than three-dimensional programs and therefore they are preferred. But the actual cases being simulated are always three-dimensional in real life, with backfill being surrounded by different conditions on each side. The plane-strain assumption used in two-dimensional models makes the approximation somewhat unrealistic. But theoretical studies have shown that stress concentrations around various symmetrically shaped openings in two-dimensional cases are always slightly larger than those prevailing in three-dimensional cases. Therefore, support performance data of consolidated backfill in symmetrically shaped openings of three-dimensional character can be safely related to the equivalent two-dimensional cases (Chen et al 1983).

There are many programs which are now available for numerical modelling in the mining industry which can apply to backfill. For example, CANMET has recently developed an elastic-plastic finite element program ("EPFE") specific to backfilling problems. This program can be easily modified to suit various applications involving backfill. A detailed example of the application of the EPFE program is given in Chen et al (1983). Here, the non-linear finite element EPFE program was used to analyze support performance of consolidated backfills of different moduli in various mining and geotechnical conditions. Cemented tailings, cemented rockfill and concrete were simulated and they were further subdivided into several types based on Modulus of Elasticity values.

There are many other types of programs that are commercially available such as ADINA and NONSAP. These however were found by Chen et al (1983) to be unsuitable for modelling backfill problems for a variety of reasons which the reader can review in the original paper. There are also other programs such as the "finite element REM (Rheological Element Model)", developed by Serata Geomechanics (see under "Serata Geomechanics" in the bibliography for reference) which can be modified for application in backfilling.

For numerical modelling of cases involving backfill, Falconbridge Limited prefers to use FLAC (a finite element difference computer modelling code) and MUDEC (a distinct element computer modelling code). These two programs were developed by the Itasca Consulting Group in Minneapolis, Minnesota and they have been used at Falconbridge since 1986. The programs have been found to be successful in simulating static and dynamic fill behavior during mining. Results using the models correlated very well with actual in-situ fill behavior. MUDEC has the added advantage of being capable of simulating the dynamic effects of blasting on backfill. A detailed account of the use of both programs in modelling backfill at Falconbridge Limited is given in a contract report sent to CANMET for publication in 1988 (see in bibliography under "Falconbridge 1988").

When employing the FEM technique on models which incorporate backfilled areas, the real-life behavior of the backfill has to be considered because the deformation modulus of backfill is lower than that of the surrounding rock (Knissel 1984). Also, many models assume that backfill is elastic in nature whereas in most cases fill exhibits plastic behavior. Programs should allow elasto-plastic and plastic assumptions to be incorporated into the model.

In accordance with the above, computer modelling to simulate and study the support capability of backfilling can take strain-softening into consideration by using a non-linear FEM analysis, as recommended by Yamatomi and Kotake (1986). This variation of the standard FEM technique allows compensation due to the fact that the mechanical properties of the backfill are so different when compared to the ore that was excavated and the rock surrounding the opening.

Input data for the non-linear FEM analysis of a spherical cavity excavated in a rock mass with strain softening characteristics is obtained from laboratory experiments on rock and cemented fill samples. Properties needed include specific gravity, Young's modulus, Poisson's ratio, compressive strength, cohesion and internal friction angle. Loading in the computer model should be represented by a uniformly distributed load equal to the unconfined compressive strength of the rock. The load is applied to the top surface of the model. Nodal forces equivalent to the gravitational body forces acting on each element should also be applied. An analysis can then be made for a case where after excavation, the cavity is filled with cemented fill.

In their non-linear FEM analysis, Yamatomi and Kotake found that instead of time-dependent deformation and mining-induced stress changes, additional vertical load should be applied step-wise up to 120 percent the full unconfined compressive strength of the rock. This compensates for time-dependent rock deformation and for the changes in the state of stress as subsequent mining activities proceed. Results of analyses with and without backfill showed considerable differences on the size of the strain-softening region indicated in the model and in the deformation of the cavity when backfill was used. The supporting effect of backfill was not immediately visible, but it appeared after the surrounding rock had deformed sufficiently and had squeezed into the filled space. Even though the mechanical properties of the fill were significantly softer and weaker than the rock (i.e. the stiffness and strength of backfill were 1/100 and 1/250 that of rock), backfill was found to have the ability to suppress progressive failure, restrict deformation of the surrounding rock and lead to improved ground stability.

There have been other interesting applications of numerical modelling of backfill. For example, numerical modelling has been used to predict vehicle mobility on a bed of dewatered mine tailings, as presented by Nowatski (1988). The computer model employed here can be modified to apply to the prediction of underground vehicle mobility over travel ways developed on backfill. From this simulation technique, vehicle mobility curves can be generated for various backfill conditions so that the best combination of degree of dewatering, curing time and tailings to cement ratio can be determined for optimal vehicle performance and economy.

Input data is critical to the success of any numerical modelling simulation program. Caution must always be used because the results can only be as accurate as the initial data on which they are based. Values for the pre-mining stress field and for rock and backfill properties must be accurately determined in order to obtain correct solutions in numerical simulation programs. The reliability of any solution obtained should be assessed by correlation with data from in-situ monitoring and instrumentation of stopes to determine whether the displacements and failure in-situ are similar to those predicted by the models. This also allows fine-tuning of the models which can be modified to better predict future cases.

CHAPTER 3

REVIEW OF BACKFILLING PRACTICE AS EMPLOYED IN VARIOUS ONTARIO AND QUÉBEC MINES

3.1 Introduction

3.2 Mining Methods

3.3 Backfill Materials
    3.3.1 Hydraulic fill
    3.3.2 Rockfill

3.4 Delivery and Placement
    3.4.1 Hydraulic fill
    3.4.2 Rock fill
    3.4.3 Special and innovative fill

3.5 Dewatering
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    3.5.2 Cut and fill stopes
    3.5.3 Fill barricades

3.6 Binders

3.7 Monitoring and Instrumentation

3.8 Backfill Problems

3.9 Backfilling Costs

3.10 Site Visit Reports

3.1 INTRODUCTION

This chapter discusses the backfill practices at a number of mines in Ontario and Quebec.

As preparation for the research work in this project a review of current backfill practices was initiated. Several site visits were conducted by Falconbridge personnel in order to assess backfill practices at various Ontario and Québec mines. During the period of July to September of 1987, a total of twelve mines operated by six different mining companies were visited. The Ontario mines visited were Placer-Dome's Dome Mine in South Porcupine, Falconbridge Ltd.'s Kidd Creek Mine in Timmins, Falconbridge Ltd's Fraser, Onaping and Lockerby mines in Onaping, INCO Ltd.'s Crean Hill, McCreedy West and Creighton Mines in Sudbury, Lac Minerals' Page-Williams Mine in Marathon, and Noranda's GECO Mine in Manitowadge and Golden Giant Mine in Marathon. In Québec, Minnova Inc.'s Lac Shortt Mine in Chapais was visited.

This chapter gives a general summary of trends and practices along with details for each mine. It should be noted that the site visits were conducted in 1987 and that some of the practices reported may no longer be employed or they may have been modified somewhat since the time they were observed. The purpose of this chapter, however, is to report the backfilling practices as they were witnessed by the personnel conducting the site visits. Updated information on backfill practices which may apply to any of the mines visited is found throughout this book.

3.2 MINING METHODS

The hard rock mines visited by Falconbridge personnel consistently used mining methods which were similar. These included blasthole stoping in both the longitudinal and transverse direction, mechanized cut-and-fill, and undercut-and-fill. There was also one case where the Avoca method was being used (Noranda's GECO mine).

In many cases, mining sequences followed a retreating panel pattern, resulting in "pillarless" mining. In this sequence, when a stope has been mined out, it is backfilled before any other activity is allowed to proceed. After sufficient time for setting of the fill, mining resumes in the stope which is immediately adjacent. In this way, no ore is left as pillars between the stopes. Such a sequence can be used in highly stressed environments to prevent the build-up of stress between stopes by eliminating the pillars completely. Typical examples were seen at the Golden Giant mine, at INCO's Creighton mine, and at Falconbridge's Kidd Creek No.2 mine.

When blasthole mining was used, the sizes of stopes varied somewhat from mine to mine, ranging from small stopes measuring 12.5 m long by 3 m wide (41 x 10 ft), to large stopes of up to 61 m long by 24 m wide (200 x 80 ft) or 80 m long by 10 m wide (260 x 33 ft). The distance between levels varied from 25 m (82 ft) to 183 m (600 ft) depending on the mine. Cut-and-fill stope sizes averaged 15 m long by 5 m wide by 3 m high (49ft x 16ft x 10ft) for conventional cut-and-fill, and 91m long by 30m wide by 6m high (300ft x 50ft x 20ft) in post pillar cut-and-fill stopes.

Slushers were still being employed in one mine while all others used load haul dump (LHD) units with bucket sizes ranging from 1.5 to 6 cubic metres (2 to 8 cubic yards). The LHD's were either diesel or electric driven and often incorporated remote control capabilities. Drilling equipment was most commonly in the hole type for blasthole stopes, with holes ranging from 114 to 165 mm (4.5 to 6.5 inches) in diameter. Single or multiple boom jumbos, including electric hydraulic units were employed for development work.

3.3 BACKFILL MATERIALS

3.3.1 Hydraulic Fill

Classified mill tailings were the most common material being used for hydraulic fill. Quarried natural sand was less common. Sand was used as a supplement to hydraulic fill when tailings supply was insufficient or by itself when tailings were unavailable or unsuitable. Tailings are considered to be a superior fill material and are used in preference when economically available.

3.3.2 Rock Fill

Rock used as fill was either underground development waste or supplied specially from surface, (quarried or reclaimed from open pit waste dumps). Surface supplied rock was commonly screened to a particular size distribution for use in backfill. This was especially so at operations using consolidated rockfill designed to stand over high exposures, e.g. Kidd Creek Mines and Page Williams Mines. Typical sizing was 75% in the range 150mm to 9.5mm and maximum of 25% minus 9.5mm. Handling the rockfill invariably resulted in attrition and a higher fines content on placement.

3.4 DELIVERY AND PLACEMENT

3.4.1 Hydraulic Fill

Hydraulic fill was usually prepared on surface and gravity-fed to underground through a network of boreholes ranging from 90mm to 115mm (3.5 to 4.5 inch) in diameter. The fill was then usually directed through victaulic pipelines to the stopes to be filled. If the stope was a large blasthole stope, a single pour point at the top of the stope sufficed. In cut-and-fill stopes, several pour points were required. In some mines (for example Falconbridge's Fraser mine), a "clothesline" was installed in the overcut prior to blasting to allow the single pour line to be pulled to any position along the length of the open stope. From there, a plug could be poured in stages, followed by the bulk backfill operation and the pouring of a working floor at the top of the filled stope, if needed.

In cut-and-fill stopes employing hydraulic fill, fill pipes were hung from the back after mining had been completed. When the filling operation took place (after installation of drainpipe and barricade components), a quantity of fill was poured at the fill fence area to stabilize the fence. Following this, pours were made from the face retreating back toward the fence.

The usual cement content for consolidated hydraulic backfill in cut-and-fill stopes was 30:1 fill to cement for the bulk fill, with a top layer of 8:1 or 10:1 for the mucking floor. A mucking floor was also sometimes provided by dumping waste in the top 1.5 to 3 m (5 to 10 ft) of the stope. The waste could then be stabilized with a mixture of cemented hydraulic fill, if desired.

Unconsolidated hydraulic fill was placed in those stopes where no adjacent mining would take place in the future. In cut and fill stopes with bulk uncemented fill, a top layer of cemented fill could be added to provide a mucking floor for the lift above. For example, at Dome mine, a 400 mm (16 inch) mucking floor having a cement content of 6:1 fill to cement was poured over stopes filled with unconsolidated fill. However, some unconsolidated fills were stable enough to allow equipment to operate on it without problem at Falconbridge's Onaping mine.

Consolidated hydraulic sand fill was used at Kidd Creek. The sand slurry at a 58% pulp density is sent underground via boreholes and 150 mm diameter steel pipe. Near the opening to be filled the sand line and a 100 mm diameter pipe carrying a 60% pulp density cement slurry join, and the mixed hydraulic fill is sent into the stope for tight filling of rock filled stopes. At INCO's McCreedy West mine, local sand was mixed with dry cement on surface and passed in dry form to mixing tanks underground where water was added. From there, the fill was pumped to the stopes. When very fine sand was used, the strength was found to be comparable to that of mill tailings fill.

3.4.2 Rock Fill

Rockfill transport methods were dependent upon the quantities of material to be used. For horizontal transport underground conveyor systems are used for the largest tonnage operations, trucks and loaders for smaller scale operations.

For consolidated rockfill, binders were most often transported to underground separately from the rock. Slurried binder mixture was added to the aggregate or rock as close to the stope as practical, generally by spraying onto the rock as it discharged from a conveyor or chute, or directly onto rock in the loader bucket or truck tray. The tumbling of the materials on dumping or falling through a baffled box ensured reasonable coating of the aggregate.

The GECO method for transport of unconsolidated rock fill was somewhat different in that it required that filling operations start prior to completely emptying the stope of its muck content. As muck was drawn at the drawpoints, fill was introduced at the top of a blasthole stope. When the material being pulled at the drawpoints changed to waste rock, the operator knew that recovery was complete. This technique greatly minimized wall sloughing\ by providing continuous support at the walls of the stope. Once the stope was entirely filled with waste, a single pour point at the top introduced 30:1 consolidated hydraulic tailings fill onto the rock until the stope was full, thereby making it "consolidated" rockfill.

3.4.3 Special and Innovative Fill

The application of paste fill was observed at the Dome mine. Underground dewatering with a tailspinner machine was used to place fill at about 75% to 80% solids by weight.

3.5 DEWATERING

Removal of excess water from backfill after placement was achieved in a variety of ways. This is mostly a concern with hydraulic fill placement.

3.5.1 Blasthole Stopes

During the site visits, the most common method for assisting in the drainage of hydraulic fills from large blasthole stopes was by simply installing drain pipes. These were usually 100 mm (4 inch) diameter perforated plastic pipes which came in customized lengths as ordered by the mine. The pipes were covered with heavy cloth which filtered out fines. Prior to filling, the pipes were lowered through the stope and pulled through the fill fence at a height of approximately 1.5 m (5 ft) above the floor. The pipes were supported by braces or secured with wire cable. They were also supported at their draining end to prevent low points from trapping water and slimes.

A high strength containment plug consisting of cemented fill with a higher than usual cement ratio was often poured to just above the brow in order to contain hydraulic backfill. The pipes would then be braced through this strong plug as well as through the fence. This was mostly done where very large fill pours could potentially produce high pressures against a fill fence.

When rock fill was being placed in large stopes, no water was needed for transport purposes. Some amount of water found its way into the fill from external sources after placement, such as normal drainage from other areas in the mine. Provisions had to be made to allow this excess water to escape from the fill area, especially when sealed fill containment barricades were being used. For example, at Kidd Creek, 100 mm (4 inch) drain lines were installed in CRF stopes for this purpose. When consolidated sandfill was used in large bulk mining stopes, small diameter longholes were drilled into the stope from above the fill fence to provide additional routes for water to drain.

3.5.2 Cut and Fill Stopes

For cut-and-fill stopes, drain sets were usually constructed on the sill areas. Decantation systems of various design were common in cut and fill stope filling. Manways in the fill were covered with filter cloth to further facilitate drainage.

3.5.3 Fill Barricades

Any structure erected for the purpose of retaining backfill in a stope is known as a fill barricade. A barricade may be a proper bulkhead, (capable of standing against 100kpa pressure as defined in chapter 4), or a simple fill fence.

The most common method of retaining large hydraulic fill pours was the containment plug. These were observed being used in the filling of blasthole stopes at many of the mines visited. A containment plug is a small pour of relatively high cement content hydraulic fill placed behind simple fill fences. The high strength fill plug acts to retain the subsequent larger, lower strength fill pours minimizing the pressure exerted against the fill fence. In this way proper bulkheads were not required for fill containment.

Plugs are poured in layers up to 5 metres thick until they extend above the brow of the drive by approximately 5 metres. Each layer is allowed to achieve initial set before the next is poured. This limits the vertical head of wet fill against the fill fence to about 5 metres, equivalent to 100kpa. Once the plug has been completely poured it was given time to achieve sufficient strength before the remainder of the stope was filled.

The containment plug is also useful as a means of securing drainage pipes at their lower end. The leakage of fines through the fill fence is minimized by having the low permeability plug act as a filter.

Typical fill fences used in blasthole stopes were either a cable fence or timber construction. When timber fences were used, wire mesh of 100 mm by 100 mm (4 inch by 4 inch) and filter cloth were placed on the active side. The filter cloth overlapped on all sides by 1.2 m (4 ft) and was usually cemented to the walls and back. The fill fences were sometimes supplemented with a catch fence to trap slimes. In wide areas to be filled, such as in longitudinal blasthole stopes, 10-ton jack posts were used to help support the standard fill fence. Where cut-and-fill stopes were being filled, fill fences were sufficient and no bulkheads were needed.

When consolidated rock fill was used, retaining barriers were still being employed. For example, "self-forming" plugs at the Golden Giant mine consisted of broken muck piled in a drift or cross-cut which acted as a combined barricade and catch fence to stop the flow of rock fill as it was being placed into the stope. The "flowability" of the rock fill could also be reduced by close monitoring of the cement content and fill density (i.e. by the addition of sand).

Pressure exerted on bulkheads and fill fences by the backfill in a stope was constantly monitored in all mines either visually or in some cases through instrumentation. When pressures were excessive, the pour was interrupted, giving time for some of the water to drain so pressure could be reduced to an acceptable level.

3.6 BINDERS

Binding agents used were either portland cement, cementitious slag, flyash or a mixture of these. Flyash was seen at Kidd Creek and Page-Williams mines. Portland cement and cementitious slags were the most common binders used. The cement ratio of the various fills varied from 6:1 or 8:1 fill to cement for working floors, to 14:1 or 16:1 for containment plugs behind the fill fence, and 20:1 or 30:1 for the bulk of the stope fill. Sand fill, with a sand to cement ratio as low as 4:1 was used at Kidd Creek for filling of interstitial voids in the stope waste. Sand fill with a much lower binder content was employed for tight filling to the back. When pozzolans or alternative binding agents were used, they replaced portland cement by up to 100 percent (as in the case of ground iron ore blast furnace slag).

3.7 MONITORING AND INSTRUMENTATION

The following list offers a summary of the testing and monitoring equipment and techniques employed by the various mines visited in connection with their backfill programs.

1. **Monitoring pressure at fill fences**
   -vibrating wire earth pressure cells
   -piezometers
   -bazooka type pressure gauges
   -visual inspection for bending or other signs of excess pressure

2. **Monitoring of pond water and drainage outflow:**
   -visual inspection
   -height measurement
   -weir calibrations
   -flow meters

3. **Fill quality and mix ratios:**
   -automatic density surge tests prior to transport to stope
   -flow meters
   -high level indicators (e.g. electric probes within tanks to give a pre-set volume)
   -electronic and manual pulp density scales
   -laboratory quality control tests
   -random sampling
   -controlled volume quantities (e.g.one slurry tankful of binder, plus one truckload of aggregate)
   -controlled weight quantities (using weightometer)
   -controlled feed time (e.g. duration for a fixed output screw type feeder)
   -sample cylinders cast or corings taken; these are subjected to uniaxial and triaxial strength tests, Schmidt hammer and vibration tests

4. **Sloughing of fill walls and dilution:**
   - -extensometers and blast vibration monitors
   - -ultrasonic borehole probes and measuring equipment
   - -surveying equipment (e.g. Wilde Distomat EDM device)
   - -sloughmeters

5. **Modelling:**
   - -computer modelling to determine required fill strength for a given free-standing height
   - -physical modelling using centrifuges

6. **Backfill feed plants:**
   - -ultrasonic high-level detectors in truck acceptance hoppers

3.8 BACKFILL PROBLEMS

There were typical problems associated with the backfill operations described. The following concerns were reported by several mines;

1. Segregation of aggregates (especially in waste rock and CTF types of fill)
2. Excessive fines at depth due to attrition in passes
3. Frozen aggregate stockpiles, causing hangups in fill raises
4. Other hangups in raises
5. Adverse effect of waste in raises on ventilation
6. Blow-out of timbered fences
7. Runs of fill at some of the self-forming plugs
8. Build-up in truck boxes, resulting in less than ideal cement ratios
9. High dilution when mining against lower cement content fills
10. High amount of slimes
11. Pipe wear and valve malfunctions due to coarse abrasive material
12. Shortage of fill materials
13. Excessive binder costs
14. High dewatering costs
15. Leakage of hydraulic fill or cement slurry in filling a stope.

3.9 BACKFILLING COSTS

Costs for backfilling mined out stopes and for erecting fill barricades have been reported by several of the mines visited under this project. Great care should be taken in comparing costs from one mine to another because of the many site specific factors to be considered, such as availability of backfilling materials, haulage distance, method of transport, depth to the stope to be filled, amount of cement required, availability of binder alternatives, etc.

Table 3.1 summarizes the costs reported at the various mines. The table shows a wide range in costs which are site dependent. The wide range in the cost of fill barricades can be attributed to the difference in cost elements considered in each case. It is not known whether some mines priced material costs only, as opposed to labour, equipment and materials. For this reason, comparison between costs for barricades should be made discriminately. It is obvious, though, that reinforced concrete construction results in a higher cost barricade than cable or timber fences. Employing a catch fence in addition to the fill fence also raises the cost.

MINE	DETAILS	TF / SF	8:1	10:1	14:1	16:1	20:1	30:1	32:1	40:1	URF	CRF	BARRICADE
Lockerby	blasthole;	4.78		14.50	11.70		9.70	8.20					7,000 to
	hydraulic fill;												10,000
	trucks on surf.												
Onaping	C&F;	5.70	17.50	15.10		11.60		8.80		8.10			
	hydraulic fill;												
	trucks on surf.												
Fraser	blasthole;	2.55	14.70	12.30		8.60	7.40		5.70	4.90			800
	hydraulic fill;												
	direct feed by												
	pipe from mill												
Kidd Creek	blasthole;										5.00 to	7.50 to	5,500 to
	hydraulic and										6.00	12.00	7,200
	trucked fill												
GECO	blasthole;	1.09						5.02			3.48		5,000
	gravity fed &												
	hydraulic fill												
Lac Shortt	blasthole;										2.50	6.72	
	rock fill only												
Page-	blasthole;											10.00	
William	rock crushed												
	on surface												

TF=tailings fill; SF=sandfill; x:1=fill to binder ratio; URF=unconsolidated rockfill; CRF=consolidated rockfill

Table 3.1 Cost per ton placed for various backfill materials and techniques (1987 Can. $)

BACKFILL PRACTICES

AT

DOME MINE

1.0 MINING METHODS

- Mechanized cut-and-fill stopes follow a series of lenses under close geological control with each lens averaging 10'H x 15'W and 50'-150'L.
- Blasthole stopes are 150'H x 20'-80'W and 100' plus L. Production now is approximately 50% from each of these two mining methods.

This mine produces 1,000,000 tonnes of ore annually recovering Au and Ag. The mine has 800 employees on site.

2.0 BACKFILL DESIGN

1) Cut-and-Fill

Ratios and Horizons - Unconsolidated hydraulic fill is used with a 16" mucking floor containing 15% Portland cement or a ratio of 6:1. The use of catwalks in these stopes has been eliminated by hanging the fill pipes from the back from the muckpile as mining progresses. When the stope is mucked out and the fill fence and drain towers are installed one pour is made at the fence to form a leakproof bulkhead and then the pours are made from the face retreating back toward the fence. Each pour is approximately 20' (breakpoint of victaulic pipe) and contains a flocculant and a hardening agent. When one pour hardens, the miner can climb on this pour, break the victaulic pipe and start the next pour. With several panels fill prepped continuous pouring is possible.

2) Dewatering Design - Drain towers are prefabricated from welded wire mesh in 24" and 36" diameters. They are braced with 2" x 2" crossbraces to maintain stability and are carefully wrapped in burlap including the top and bottom of the tower. A piece of 1/8" plywood at the bottom of the tower contains a hole for a 3" plastic pipe which runs from the drain tower along the floor and through the fill fence for dewatering. This pipe is covered with burlap and weighted down with muck to prevent floating. The number and size of the towers vary with length and width of each pour.

Conventional filling methods in cut-and-fill stopes are also carried out.

3) Bulkhead Design - Fill fences consist of 6" vertical lagging sunk into the previous fill floor with 1" x 6" planks nailed horizontally to these lagging with 3" spacings between the planks. Burlap is stapled

to the planks on the fill side with a 3′ overhang on the floor and on each wall. The burlap is weighted down on the floor and well cemented to each rock wall. Most of the timber is salvaged after completion of the pour.

4) Pour location and Design - Pour locations as described in (a) are predetermined although usually at 10′ or 20′ intervals (standard victaulic pipe lengths). This pouring method eliminated costly and somewhat hazardous catwalks. It is also a more economical and faster method if organized properly.

3.0 BACKFILL COSTS

1) Cost Per Ton	6:1 Unconsolidated	Classified	Average combined $4.00/Ton

2) Bulkhead Fences - Cost unavailable

3) Bulkhead Stope Prep. - Cost unavailable

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods - The method of backfilling a cut-and-fill stope as described above, while not a completely new idea, is working very well in some areas of the mine. Other stope crews and supervisors have as yet not accepted this method, however with further experience it would appear that it will eventually become a standard procedure.

2) Monitoring - The fill plant is fully instrumented and cement ratios and pulp densities are indicated by digital readouts and can be set automatically and adjusted by the operator. 6:1 consolidated fill is sent directly to the stopes while classified tailings is sent to two underground storage tanks where it is repulped and pumped to the stopes.

Where bulkheads were used, piezometers were installed. If pressure readings reached 6 p.s.i. pouring was stopped until the readings subsided.

5.0 SHRINKAGE AND BLASTHOLE VOID FILLING

Backfill Design

1) Fill Ratios and Horizons - Designing bulkheads for the many openings at the bottom of the large voids at this mine would have been in the best case highly expensive and in many cases impossible due to the inaccessibility of the openings. It was therefore necessary to find

a fill capable of forming its own bulkheads or plugs and through a series of experiments it was proven that a consolidated paste fill could be formed to meet these criteria. A complete description of the background work, test results and conclusions can be found in Dale Churcher's paper of April 1986 - "Evaluation of High Density Backfill System at Dome Mines".

A brief description of the process for forming the paste fill is as follows:

Tailings is recovered from the mill at a pulp density of approximately 62% and is transported to a repulper tank and then to a tailspinner surge tank at a density of 68%. The material to be used as dense fill passes through a vibrating screen to remove wood chips and enters a tailspinner conditioning tank. $FeSO_4$ is introduced into this tank to reduce the free cyanide levels to an acceptable limit. Automatic metered water valves lower and maintain the pulp density at 58%. The feed material is pumped underground into surge tanks. From these tanks the slurry is pumped into tailspinner centrifuges for dewatering. The heavy product (paste) is dropped into a mixing screw conveyor which transfers it into a second mixing screw conveyor and the paste is deposited directly into the void at a density of 77%. A 50-50 mixture of cement and flyash is fed by a metered conveyor into the first mixing screw conveyor. The second mixer ensures a full coating of the fill with the additive. The ratios vary from 25:1 to 40:1 as required. Three tailspinner centrifuges will allow 800 tons of fill to be poured in a 24 hour day. Sixteen tons of slurry enter each centrifuge and thirteen tons emerge as paste fill. The remaining three tons are fines and are rejected with the water due to the nature of the centrifuge and are pumped to surface.

Note: The paste fill produced may also be pumped using a concrete pump to areas several hundred feet from the underground tailspinner station.

2) Dewatering Design - The paste fill has very little excess water and the percolation rate is near zero. Any influx of ground water will seep along the footwall.

3) Bulkhead Design - The angle of repose of the paste fill at 25:1 is approximately 62° and forms a natural bulkhead. This initial pour forms a plug which is economical and which will resist liquification under normal mine conditions.

Concrete bulkheads were formerly used in shrink stopes and varied from 3' to 4'6" thick. They were designed to withstand 300' of water head.

4) Pour Location Design - The paste fill drops vertically from the head of the screw-conveyor resulting in a fill spreading problem The flowability of the paste depends upon the cement content and the

moisture content and ongoing experiments to improve this situation are being carried out. Innovations such as flow aids and throw conveyors are being considered.

It should be noted that the voids at Dome are so large that a single pour point may be in place for three years.

6.0 BACKFILL COSTS

Paste fill $5.00 per ton placed.

7.0 INNOVATIVE BACKFILL PRACTICES

1) Methods - The use of paste fill is a recent innovation in Canada, the concept having been imported from South Africa.

   Dome Mines is currently working improving all aspects of paste fill placement and Mr. Churcher of Placer Dome Mines and CANMET are carrying out a joint research project to arrive at a successful method of coring and other testing of in situ material.

2) Monitoring and Instrumentation

   i) Instrumented water inflow valves to keep density of slurry going underground at 58% density although it cannot increase density.

   ii) In underground surge tanks at the tailspinner station the slurry is automatically metered to feed each centrifuge at 75 U.S.G.M.

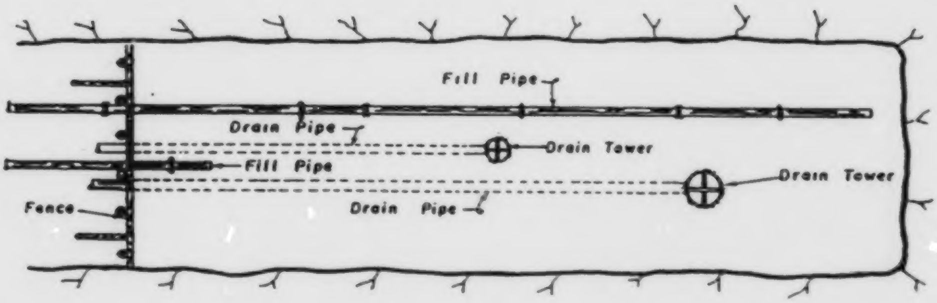
   iii) The cement-flyash binder is automatically metered into the screw conveyor.

   iv) Lab tests for compressive strengths, percolation rates, ratios and moisture content are carried out. (Note: 28 day Uniaxial compressive strength of paste fill is 100 p.s.i.).

Rockfill

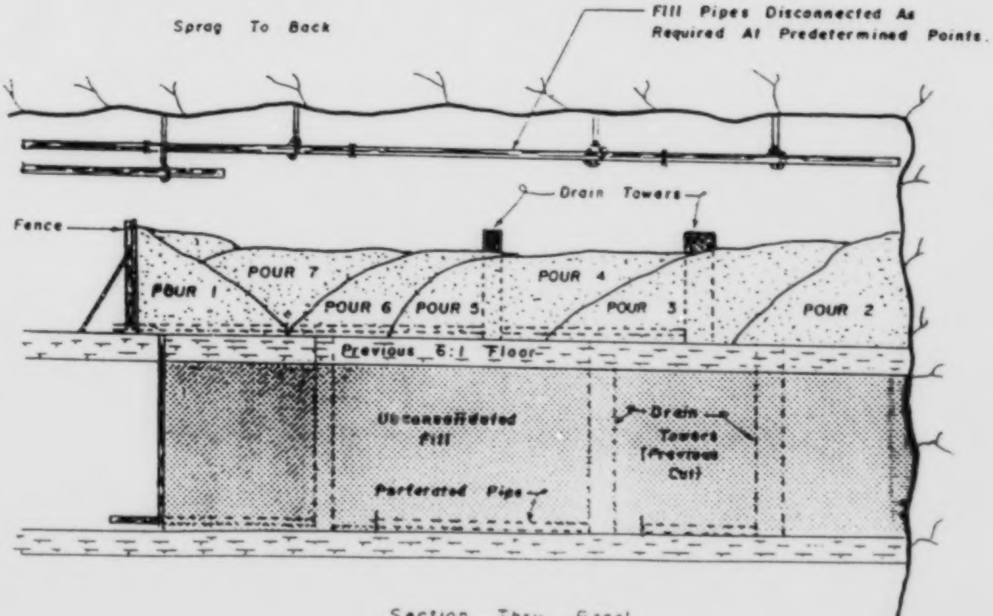
Rockfill (up to 1,000,000 tons per stope) will soon be placed in certain voids from surface. Alimak and drop raises have been completed to pass the rock from surface into the void. Waste rock is in short supply at Dome and they are planning to start their own rock quarry.

Note: Each Drain Pipe To Fence Individually.


Fill Pipe
Drain Pipe
Drain Tower
Fill Pipe
Drain Tower
Fence
Drain Pipe

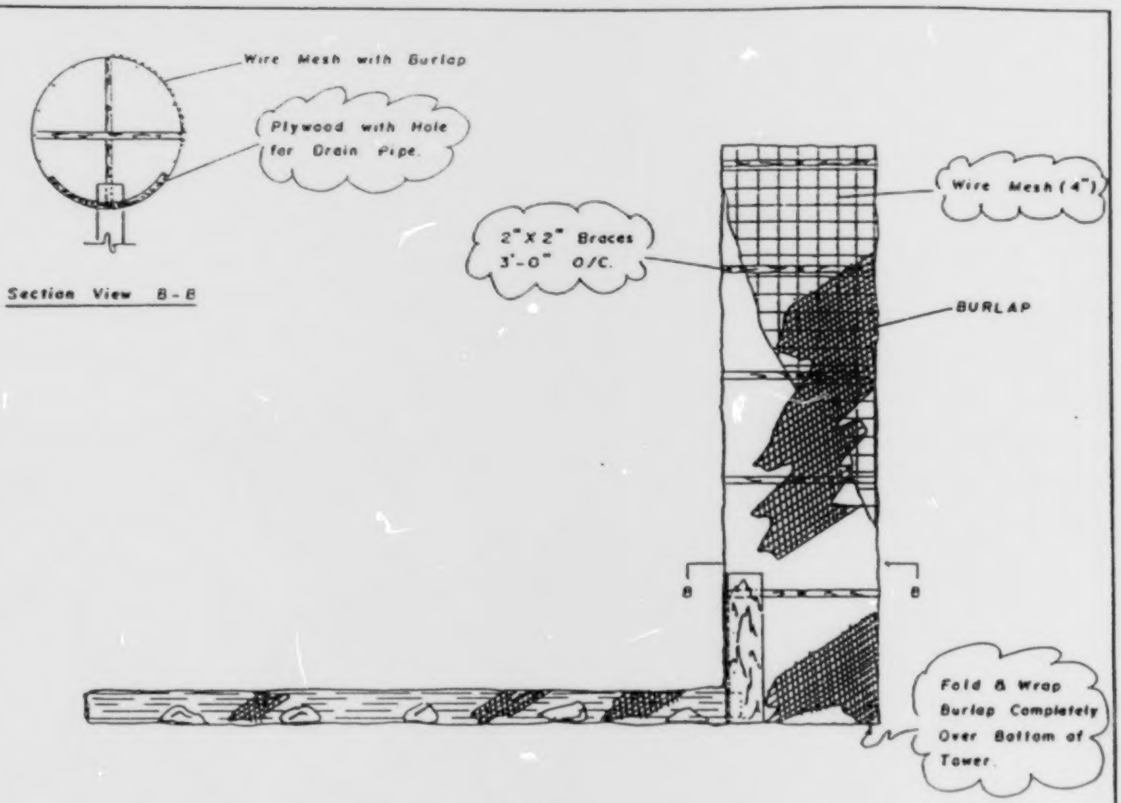
Plan of Panel

CUT and FILL STOPE FILL POUR


Sprag To Back
Fill Pipes Disconnected As Required At Predetermined Points.
Drain Towers
Fence
POUR 7
POUR 1
POUR 6
POUR 5
POUR 4
POUR 3
POUR 2
Previous 5:1 Floor
Unconsolidated Fill
Drain Towers (Previous Cut)
Perforated Pipe

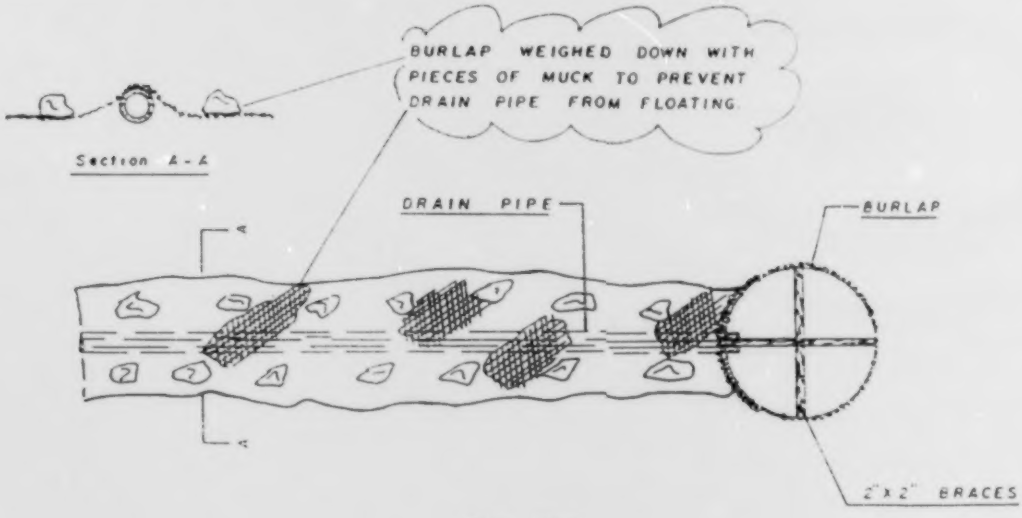
Section Thru Panel

DOME MINE


Wire Mesh with Burlap
Plywood with Hole for Drain Pipe.
Section View B-B
Wire Mesh (4")
2" X 2" Braces 3'-0" O/C.
BURLAP
B
B
Fold & Wrap Burlap Completely Over Bottom of Tower.

Section

24" or 36" Drain Tower C/F Stope


BURLAP WEIGHED DOWN WITH PIECES OF MUCK TO PREVENT DRAIN PIPE FROM FLOATING.
Section A-A
DRAIN PIPE
BURLAP
A
A
2" x 2" BRACES

Plan View

DOME MINE - DRAIN TOWER

BACKFILL PRACTICES

AT

FRASER MINE

1.0 MINING METHODS

Fraser Mine is presently producing 605,000 tonnes of ore annually and will reach 1,000,000 tonnes per year at full production. The metals recovered are Ni @ 1.6% and Cu @ 0.80%. Mining is by blasthole stoping drilling sub-level to sub-level (30 m) with 6½" diameter I.T.H., Rand and Mission drills and mucking remote with ST8 scooptrams. The average stope dimension is 80 m x 10 m x 30 m (L.W.H.).

2.0 BACKFILL DESIGN

1) Ratios and Horizons

Classified tailings fill is obtained from Strathcona Mine through two 3" pipelines, gravity fed from the 2000 level at Strathcona to the 2250 level at Fraser Mine.

Fill ratios are 16:1 for fill plugs to 1 meter above the brow and 32:1 or straight fill depending on whether it is a primary, secondary or tertiary stope. Five to ten feet of 16:1 waste fill provides a mucking floor at the top of each stope.

Portland cement and slag are the binding agents used and up to 50% of the cement may be replaced by slag in the pour above the plug. Sand may also be added to this pour in ratios up to 15%.

2) Dewatering Design

Three 4" drain lines are hoisted from the undercut to the overcut. These fabrene covered pipes are then passed through the fill fence approximately 1.5 m above the floor and are also supported by x-braces to eliminate low points. Two small catch fences are usually installed.

3) Bulkhead and Fill Fence

A timber fill fence is used at this mine (see attached dwg.). On the active side of the fence, 4" x 4" wire mesh and fabrene are placed with the fabrene overlapping all sides by 4 feet and cemented to the walls and back.

4) Pouring Design

A clothesline is installed in the overcut prior to blasting and enables the single pour line to be pulled to any position along the length of the open stope.

3.0 BACKFILLING COSTS

1) Cost Per Ton of Fill Placed

Ratios	S.F.	8:1	10:1	16:1	20:1	32:1	40:1
	$ 2.55	$14.70	$12.30	$8.60	$7.40	$5.70	$4.90

2) Fill Fence Cost

Material	$ 187.00
Labour	574.46
Total	$ 761.54 or 2.89 sq. ft.

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

Installations of clothesline in overcut allows even pouring of the fill into the open stope.

2) Monitoring and Instrumentation

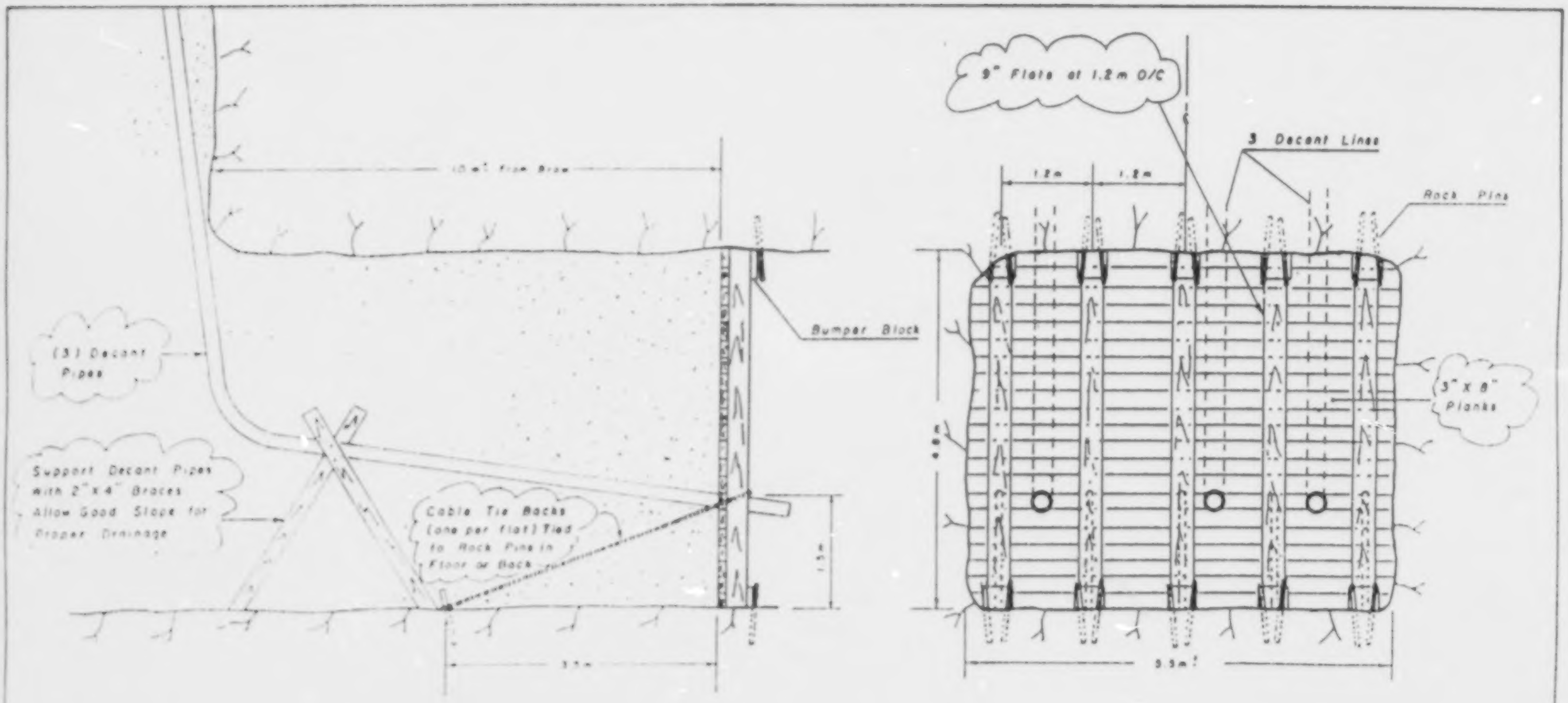
Pressure gauges are installed on the fences and are checked each pouring shift by the shift foreman. The foreman also measures the water flow and checks for ponded water.

Fraser Mine issues a detailed fill layout and a record form which must be filled out each shift by the foreman.

A Wilde Distomat has been purchased to check free standing fill walls to monitor dilution.

5.0 EXISTING PROBLEMS

1. High amount of slimes.
2. Dilution is high against 32:1 fill.


9" Flats at 1.2m O/C
3 Decant Lines
1.2m
1.2m
Rock Pins
3" X 8" Planks
4.8m
5.5m±
10 m± from Brow
Bumper Block
(3) Decant Pipes
Support Decant Pipes with 2" X 4" Braces Allow Good Slope for Proper Drainage
Cable Tie Backs (one per flat) Tied to Rock Pins in Floor or Back
1.5m
3.5m

Side View

Front View

Standard Blast Hole Stope Fill Fence

Fraser Mine

BACKFILL PRACTICES

AT

GECO MINE

1.0 MINING METHODS

Geco Mine is currently producing approximately 1 1/2 million tons of ore recovering Cu, Pb and Zn. They are presently in a pillar recovery stage and are developing a deep zone with a switchback ramp. The predominant mining method is blasthole (Geco method) mucking with 125 h.p. slushers in scram drifts. The sizes of the stopes are 100' x 70' x 200' - 400' (LxWxH). Some Avoca type stopes are being mined.

2.0 BACKFILL DESIGN

1) Ratios and Horizons - The tailings fill plant area has a 200 ton cement silo, two mixing tanks and one surge tank. The cement ratio is obtained by setting the weightometer scale to a calculated cement to classified tailings fill ratio of 1:30. A screw conveyor feeds the cement into the mixing tank. Cement is the only additive presently used. Density of the mixture is 64% and is checked with a Marcy scale.

   Surface waste from a quarry is passed through three fill raises. Finger raises to these main raises feed waste directly to stopes. The waste is uncrushed but well fragmented.

2) Dewatering Design and Bulkheads - A standard reinforced concrete bulkhead (5' thick) is installed at each opening into the stope prior to pouring the consolidated tailings fill. Over 1000 of these bulkheads have been installed at Geco. Drain lines are installed with a mousetrap on the active side.

3) Pour Location and Design - When blasting of the stope is underway and mucking has started a void will develop at the top of the stope. Waste is now passed through a finger raise which has been driven directly over the stope. As the void fills the raise will choke off until more ore is pulled. The waste continues to fill the void as it is created by mucking from the bottom of the stope and provides support keeping wall slough to a minimum. When the stope is emptied of ore it will be full of waste and if it is a pillar this will be the end of the filling cycle. If stopes are to be mined adjacent to this block then a single pourpoint at the top of the stope will introduce 30:1 consolidated tailings fill into the waste rock. Pouring continues until the stope is full. The standard bulkheads are installed prior to pouring the C.T.F.

3.0 BACKFILLING COSTS

Hydraulic Fill	$ 1.09/ton placed
30:1 C.T.F.	$ 5.02/ton placed
Waste Rock (quarry)	$ 3.48/ton placed
Bulkheads	$ 5,000.00 each

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

Natural flow of waste into stopes reduces handling.

Geco is considering using slag from Agloma Steel to reduce cement usage.

2) Monitoring and Instrumentation

Bulkheads have "bazooka" type pressure guages and pressures are carefully monitored during fill cycle.

The fill plant has a weightometer on the cement feeder to ensure the proper ratio of cement to fill.

Warning devices in the fill plant sound if some part of the cycle malfunctions. This allows the operator to do work in other areas of the mill.

5.0 PROBLEMS ENCOUNTERED

Segregation - both of waste and C.T.F.

Hangups in raises

Ventilation changes depending on the waste in the surface raises.

Too many fines at depth due to attrition in passes.

BACKFILL PRACTICES

AT

GOLDEN GIANT MINE

1.0 MINING METHODS

The Golden Giant Mine is scheduled for a production rate of one million tonnes annually. MacIsaac Mining is currently developing the deep zone of the mine. The mining method is blasthole stoping blasting into slot raises. These raises are dropped using 4½" diameter I.T.H. drills for the cut and 2-1/8" diameter holes for the square. A retreating panel sequence is now being used rather than leaving pillars. The average stope size in the upper zone is 100 m x 25 m x 20 m (HxLxW) while in the lower zone the average dimensions are 50 m x 15 m x 20 m (HxLxW). There are 275 employees at Giant.

2.0 BACKFILL DESIGN

1) Ratios and Horizons - Consolidated rock fill is used at this mine with cement as the only binding additive. The fill plant has one cement silo with 150 tonne capacity and also has sand silos which have not been used as yet. Cement ratios are approximately 3 to 4% but in the past ratios of up to 10% have been used. Cement is added as preset on a computer and is automatically monitored to stop when S.G. of slurry reaches 1.77 which corresponds to a density of 64%.

   Surface waste from a quarry is crushed to minus 8" and passes underground through fill raises. Development waste is occasionally added to the crushed rock on a 50-50 basis. When this is done the crushed rock is screened to remove all fines below 3/4". This is due to the nature of the schist waste which tends to provide excessive fines.

2) Dewatering Design and Bulkheads - Self-forming plugs are formed by piling muck in drift or x-cut to act as a catch fence to stope the flow of C.R.F. as it pours out of the stope. The angle of repose of the fill is about 33°. The cement ratio is 4% but close monitoring by the operators can have the amount of slurry reduced to raise the fill density and lower the flowability. The small percentage of water in the fill percolates from the fill and flows to sump areas.

3) Pour location and Design - All filling at Giant is currently done by dumping the C.R.F. directly into the stope or through a drop raise. Fill trucks holding 17 tonnes of consolidated fill are used.

3.0 BACKFILLING COSTS

Hydraulic Fill	$ 1.09/ton placed
30:1 C.T.F.	$ 5.02/ton placed
Waste Rock (quarry)	$ 3.48/ton placed
Bulkheads	$ 5,000.00 each

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

Natural flow of waste into stopes reduces handling.

Geco is considering using slag from Algoma Steel to reduce cement usage.

2) Monitoring and Instrumentation

Bulkheads have "bazooka" type pressure gauges and pressures are carefully monitored during fill cycle.

The fill plant has a weightometer on the cement feeder to ensure the proper ratio of cement to fill.

Warning devices in the fill plant sound if some part of the cycle malfunctions. This allows the operator to do work in other areas of the mill.

5.0 PROBLEMS ENCOUNTERED

Segregation - both of waste and C.T.F.

Hangups in raises.

Ventilation changes depending on the waste in the surface raises.

Too many fines at depth due to attrition in passes.

BACKFILL PRACTICES

AT

INCO METALS

1.0 MINING METHODS - CREIGHTON MINE

This mine is producing 9350 tons of ore per day on a 5-day week, recovering 1.46% Ni and 1.15% Cu. The mining methods are vertical crater retreat blasthole, undercut and fill and mechanized cut-and-fill. Blasthole stopes are normally 40′ W x 40′ L with 200′ level intervals. Drilling is level to level with 6½" diameter I.T.H. drills. Mining sequence is to take the centre panel and retreat to both ends to reduce stress buildup and lower the risk of rock bursting. This means that all panels must be filled with consolidated fill. Mucking is with 5 cu. yd. electric scoops with remote control. Undercut and fill stopes are taken in 13′ wide panels 12½" high, leaving two panel widths between initial stopes.

2.0 BACKFILL DESIGN

At Creighton Mine Portland cement has been replaced with Watts slag. This slag is granulated at Algoma Steel and transported to Spragge where it is reslimed and ground in ball mills to a fine powder and then transported to Creighton Mine in trucks which blow the slag into a silo at the fill plant.

In the fill plant, slurried lime is added to the slag and classified tailings on a weight basis with the slag binder. The amount of slag binder is added on the same basis as Portland cement and the cost is 30% less. The lime speeds up the setting time of the fill.

1) a) U.C.F. Stopes - In these stopes 2′ of 12:1 is poured to bring the fill just over the laminated beams, then 4′ to 6′ of 20:1 is poured and the remainder 30:1 and straight fill.

   b) Blasthole Stopes - A 15:1 plug is poured in several stages to 15′ above the brow and the remainder of the stope is poured with 30:1.

   c) Mechanized cut-and-fill stopes - 30:1 F.T.C. with a 10:1 mucking floor.

2) Dewatering Design

In blasthole stopes four drain lines, consisting of 4" flexible perforated pipes covered with Paddock cloth, are lowered from the top of the stope and hang down the near wall (see drawing). They are then pulled through the draw drift and pass through the timbered fill fence. The top of the pipes are secured to the wall of the top sill with a wire cable.

3) Bulkhead Design

There are no bulkheads in Creighton Mine. Timbered fill fences with wire mesh and fabrene on the active side are installed (see attached drawing). There is no standard fill fence at Creighton.

In the underground backfill station the truck driver activates the batch tank pump and fills a controlled volume slurry tank. When the slurry tank is full he parks his truck box under an aggregate chute and activates controls which starts the flow of aggregate and opens the valve from the slurry tank. The aggregate now flows into the truckbox and is sprayed by the flow of slurry. The driver moves his truck back and forth to even the load and improve mixing.

3.0 BACKFILLING COSTS - NOT AVAILABLE

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

Self forming plugs with C.R.F. provides and efficient and reliable bulkhead.

Mixing of development waste with crushed rock.

2) Monitoring and Instrumentation

Close control and computerization in the fill plant as well as sampling provide an accurate ratio of cement and control the density of the slurry going underground.

In the backfill station underground there is no instrumentation control as such but the basic approach of using the controlled volume slurry tank to combine with one truckload of aggregate appears to work quite well. There are no lab facilities at this mine and samples are sent to Canada Cement for testing.

5.0 PROBLEMS ENCOUNTERED

i) Segregation of fill.

ii) Blow out of timbered fences.

iii) Runs of fill at some of the self-forming plugs.

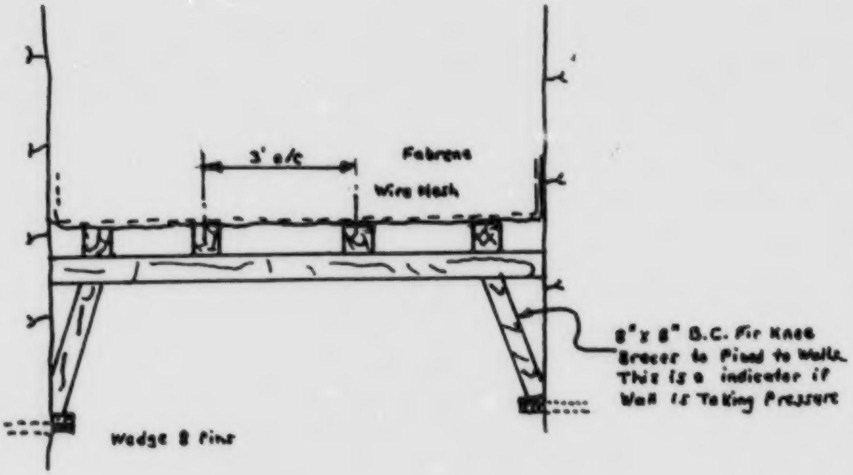
iv) Control of ratio due to buildups in truck box resulting in less aggregate per load and higher cement and rates.

Note: A 50 m free standing fill was observed at this mine which appeared to have very little spalling or other failure.

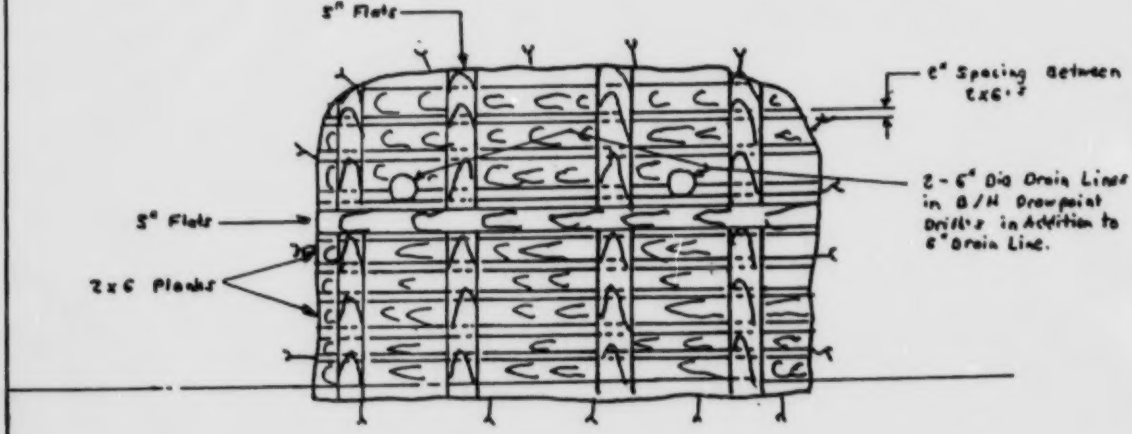
6.0 LEVACK-MCCREEDY WEST

At these mines there is no source of tailings fill so local sand is used and mixed with dry cement on surface and passed to mixing tanks underground. Previous tests indicated C.S.F. to have only 75% of the strength of equivalent C.T.F., however with the present use of a very fine sand the results of strength tests are now comparable.

A test plant was set up at Levack Mine for the production of high density paste fill. A mixture of repulped tailings, aggregate and Portland cement produced a 90% density fill which was pumped underground using a concrete pump through an 8" line to one underground test stope. An undercut and fill stope will be mined under this fill.


3' o/c
Fabrene
Wire Mesh
8" x 8" B.C. Fir Knee Braces to Pinned to Walls. This is a indicator if Wall is Taking Pressure
Wedge & Pins

Top View


5" Flats
2" Spacing Between 2x6's
2 - 6" Dia Drain Lines in B/H Drawpoint Drill's in Addition to 6" Drain Line.
5" Flats
2x6 Planks

Front View

INCO - Creighton Mine

Fill Fence

BACKFILL PRACTICES

AT

KIDD CREEK

1.0 MINING METHODS

Kidd Creek produces 4.335 million tonnes of ore annually, recovering Cu, Pb, Zn and Ag. The principal mining method is open stope blasthole (98%) with stoping sizes of 50′ - 200′ L, 50′ - 80′ W and 100′ - 600′ H.

2.0 BACKFILL DESIGN

Combinations of rock, waste and sand fills are used at this mine. Only consolidated rockfill and consolidated sand fill will be discussed in this summary.

a) Fill Ratios

C.R.F. (consolidated rockfill) which is an aggregate crushed to 75% 3/8" to 6" and 25% less than 3/8". The aggregate contains a considerable amount of fines which increase with transfer through a waste pass to the underground operations. Portland Cement and flyash (Type C) are added to the aggregate in amounts from 4 to 6%. Higher additive ratios are used for the first pour (plug) and for higher free-standing requirements. These pours are calculated by the backfill engineer and issued to the operators. Flyash may replace cement up to 65%, resulting in a considerable cost reduction.

The cement-flyash is mixed in the fill plant in two mixing tanks, each holding 11 tonnes of additive and 2500 U.S. gallons of water (maximum). The slurry is sent underground through 7 7/8" boreholes and then through 4" and 6" victaulic pipes at a pulp density of 40-60%.

C.S.F. (consolidated sand fill) is used for tight filling and has an additive ratio of from 3.5% to 25% depending on adjacent mining requirements. The sand is introduced to the underground workings as a slurry (50% density) and mixed with slurried additives near the pour point by joining of the two pipelines.

3.0 DEWATERING DESIGN

Each batch (one mixing tank) of the additive in slurry form is mixed with approximately 200 tonnes of aggregate which results in a low water to solid fill ratio, thereby lessening drainage problems.

C.S.F. introduces more drainage problems due to the higher water content. To offset this problem 2 1/8" long holes are drilled above the bulkheads into the stope to facilitate drainage.

1) Bulkhead Design

The standard bulkhead design is a 14" thick reinforced concrete wall at each drawpoint. It is designed for 5000 p.s.i, and has two 4" drain pipes through it to remove water from behind the bulkhead. The pipes have a mousetrap to prevent fines from building up in the drift.

2) Pour Location Designs

The aggregate is passed from surface to the 8(2A) level (180 m) where a vibrating feeder loads a permanent 30" conveyor with controlled amounts of fine and coarse material. This aggregate is conveyed to a shuttle which feeds either a raise to #1 shaft or #2 shaft, as required.

The permanent conveyor transfers the aggregate onto a series of portable conveyors which transport the material to the open stope.

The slurried batch of additive is piped from surface to skid mounted mixing tanks near the pour sites and is pumped through 4" or 6" victaulic pipelines which parallel the portable conveyors. At the open stope the aggregate is sprayed with additive as it falls from the head of the conveyor into a baffled chute or 4"0 steel pipe which ensures full coating of the aggregate. The sprays simply consist of three 2" pipes which are welded into the capped slurry line.

A second method is to have Jarco 6.4 tonne trucks load aggregate from loadout points and spray each truck load before dumping it into the open stope. Other variations of mixing are also used depending on specific requirements and accessibility.

C.S.F. is usually introduced into the stope through 18" boreholes drilled and reamed from the level above into the stope for tight filling.

4.0 BACKFILLING COSTS

1) Backfilling, $/Ton Placed

#1 Shaft 40% Cement, 60% Flyash   cement only
#2 Shaft 65% Cement, 35% Flyash

#1 Shaft	#2 Shaft	Ratio %	#1 Shaft	#2 Shaft
		0	$ 5.00	$ 6.00
$ 7.50	$ 8.60	3.0	8.00	9.00
7.90	9.00	3.5	8.50	9.50
8.30	9.50	4.0	9.00	10.00
9.10	10.30	5.0	10.10	11.00
9.90	11.20	6.0	11.10	12.00

2) Bulkhead Costs

Conc. Bulkhead #1 Shaft $ 5,538.00 = $ 33.00 sq. ft.
" " #2 Shaft $ 7,203.00 = $ 43.00 sq. ft.

Timber Fence - Few used, no standard, no cost available

5.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods - i) slag had been replaced by flyash

ii) experimenting with after pour strengthening (grouting)

iii) experimenting with monolithic packing to replace C.S.F. for tight filling to the back

iv) introducing flocculants to C.S.F.

v) using retarders on last two feet or individual pours to prevent impact damage from subsequent pour

2) Monitoring - i) Surface - in the fill plant mix ratios are controlled by the operator using flow meters, high level indicators and electronic and manual pulp density scales.

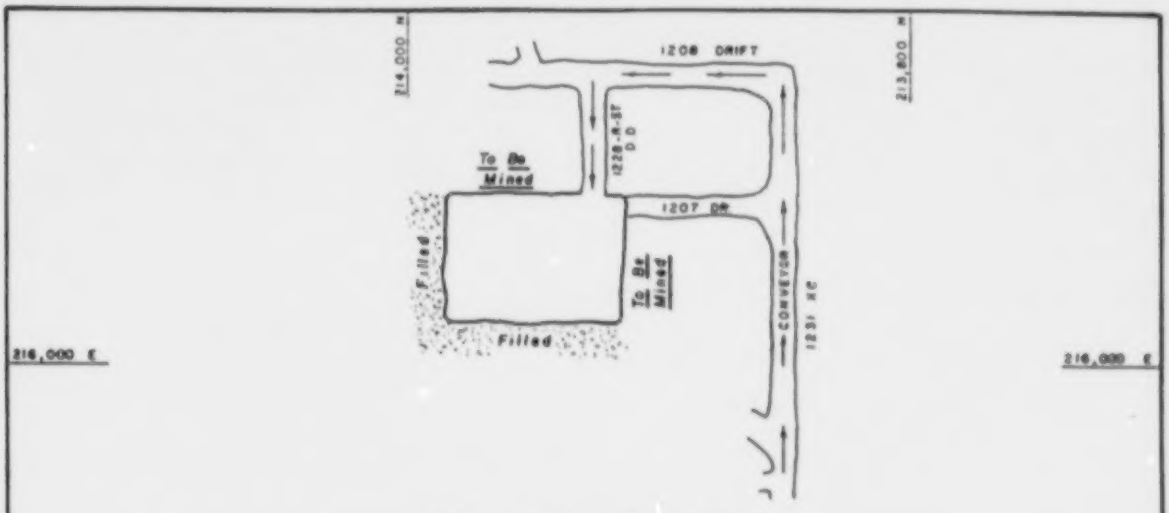
In the laboratory extensive quality control tests are continually carried out in an effort to maintain required fill strengths to minimize dilution problems.

3) Underground One operator controls the amount of aggregate using a digital readout from a suspended weightometer and the amount of slurry again from a digital readout from a flowmeter.

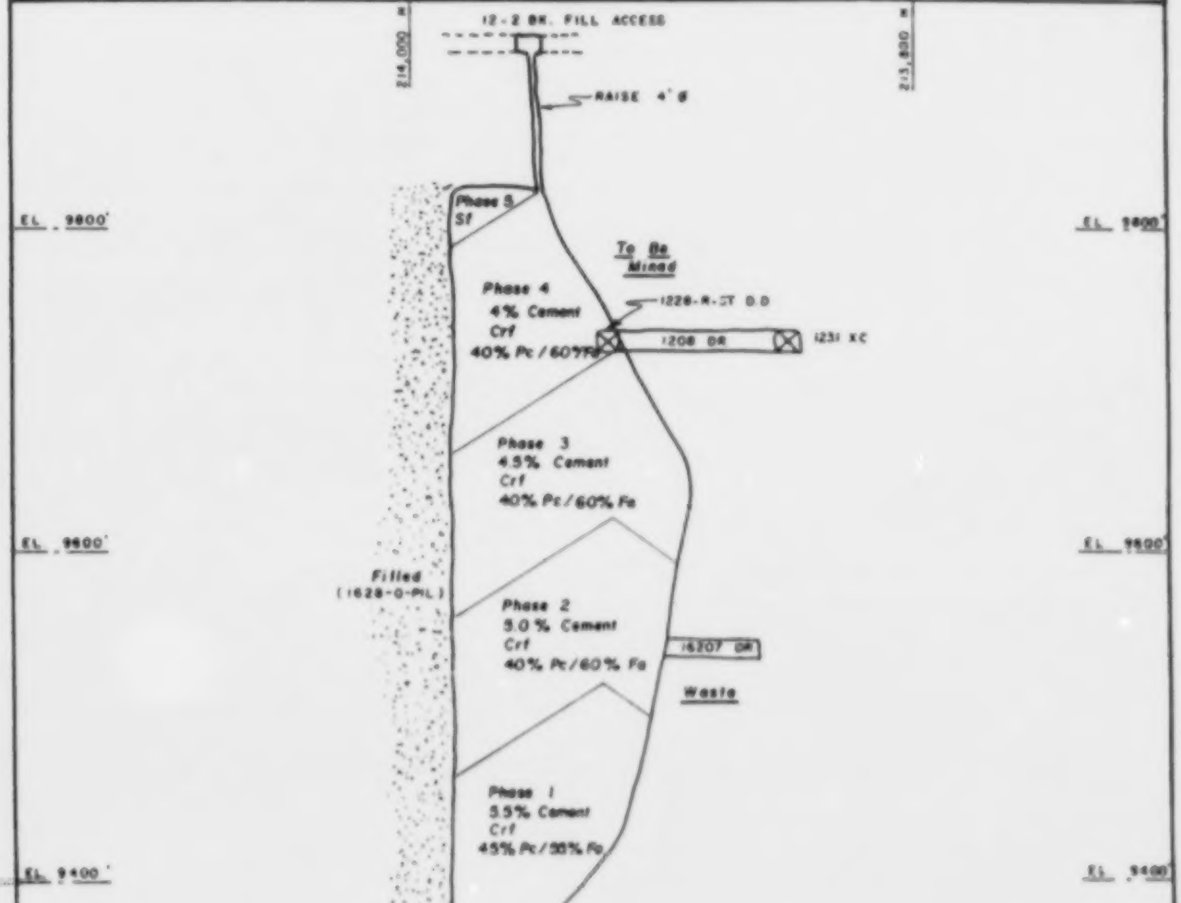
A batch to tonnage ratio is also closely observed to follow instructions on the fill sheet issued by the backfill engineer.

Test drifts have been drilled and blasted through C.R.F. to observe results and to carry out tests on the drift walls, such as coring, Schmidt hammer tests for compressive strength and effects of vibration.

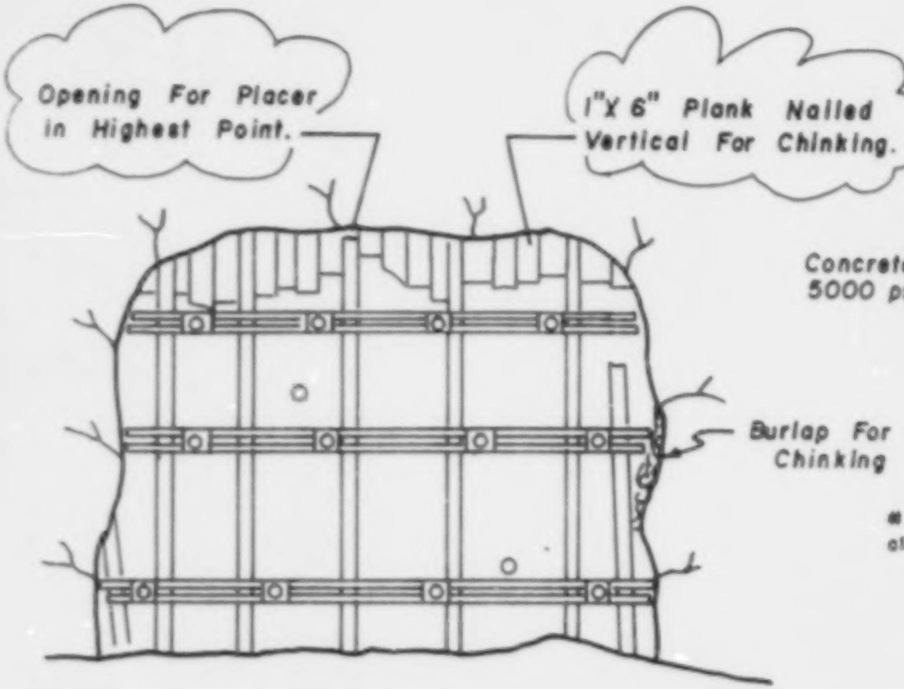
There is a slurry measuring tank for each truckload of aggregate.


214,000 N
213,800 N
1208 DRIFT
1228-R-ST D.D
To Be Mined
1207 DR
Filled
To Be Mined
Filled
CONVEYOR
1231 XC
216,000 E
216,000 E

FILL PLAN- 1200 LEVEL
1629-Q- PIL

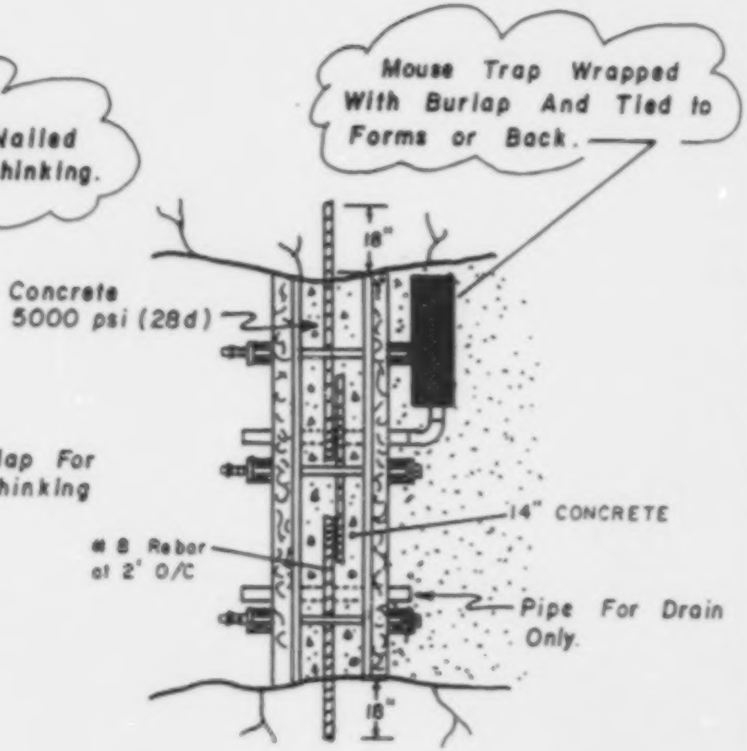

214,000 N
213,800 N
12-2 BK. FILL ACCESS
RAISE 4' Ø
EL. 9800'
EL. 9800'
Phase 5
Sf
To Be Mined
Phase 4
4% Cement
Crf
40% Pc / 60% Fa
1228-R-ST D.D
1208 DR
1231 XC
Phase 3
4.5% Cement
Crf
40% Pc / 60% Fa
EL. 9600'
EL. 9600'
Filled
(1628-Q-PIL)
Phase 2
5.0 % Cement
Crf
40% Pc / 60% Fa
16207 DR
Waste
Phase 1
5.5% Cement
Crf
45% Pc / 55% Fa
EL. 9400'
EL. 9400'
1629 XC

ELEVATION VIEW- FILLING PHASES.
— KIDD CREEK MINE —


Opening For Placer in Highest Point.
1" X 6" Plank Nailed Vertical For Chinking.
Burlap For Chinking

Front View

Note: Use Recycled Timber When Available.


Mouse Trap Wrapped With Burlap And Tied to Forms or Back.
18"
Concrete 5000 psi (28d)
14" CONCRETE
# 8 Rebar at 2' O/C
Pipe For Drain Only.
18"

Side View

—KIDD CREEK MINE—

STANDARD BULKHEAD

BACKFILL PRACTICES

AT

LAC SHORTT MINE

1.0 MINING METHODS

Lac Shortt Mine produces 400,000 tonnes of ore annually at a grade of 4.5 to 5.0 grams of Au per ton. The stoping method is longitudinal blasthole, mucking with 5 yd scoops in draw points originating at a footwall drift.

The ore body varies from 10′ to 25′ wide with primary stopes being 12.5 m along strike and pillars 25 m on strike. Drilling is with 6½" diameter I.T.H. drills from level to level which is 50 meters.

2.0 BACKFILL DESIGN

1. Fill Ratios and Horizons

Consolidated rockfill is used in the primary stopes consisting of development waste with a 55% pulp density slurry of water and Portland cement added to the waste. A ratio of 3% cement is used in primary stopes and 6% is poured for a height of 10 meters where horizontal sills are left.

Waste alone is dumped into the pillar stopes with the addition of sand if waste is in short supply.

2. Dewatering Design and Bulkheads

Bulkheads are not used at this time. Cable fences are installed to hold back the fill until a plug has been established (see attached drawing) of standard fill fence.

3. Pour Location and Design

In the backfill station of the level requiring fill a slurry batch from the fill plant is gravity fed into a mixing tank. The waste pass system also has a finger raise into the backfill station with an air operated cylinder to control the chute gate. A measuring tank is filled with slurry to a present level controlled by an electric probe. The level in this tank varies with the required ratio of cement to waste. A JDT415 truck with an actual load capacity of 9 tonnes is positioned under the waste chute and a pipe from the measuring tank. The truck operator activates the control chute to fill the truck box and opens the valve from the measuring tank allowing the cement slurry to mix with the waste (see attached drawing). The load is then hauled to an open stope and dumped from the top of the stope.

3.0 BACKFILLING COSTS

1.	Fill Placed	- Consolidated waste fill (aver.)	$6.72/tonne
		- Waste fill	$2.50/tonne
2.	Fill Fence	- Cost not available	

4.0 INNOVATIVE BACKFILLING PRACTICES

1. Methods

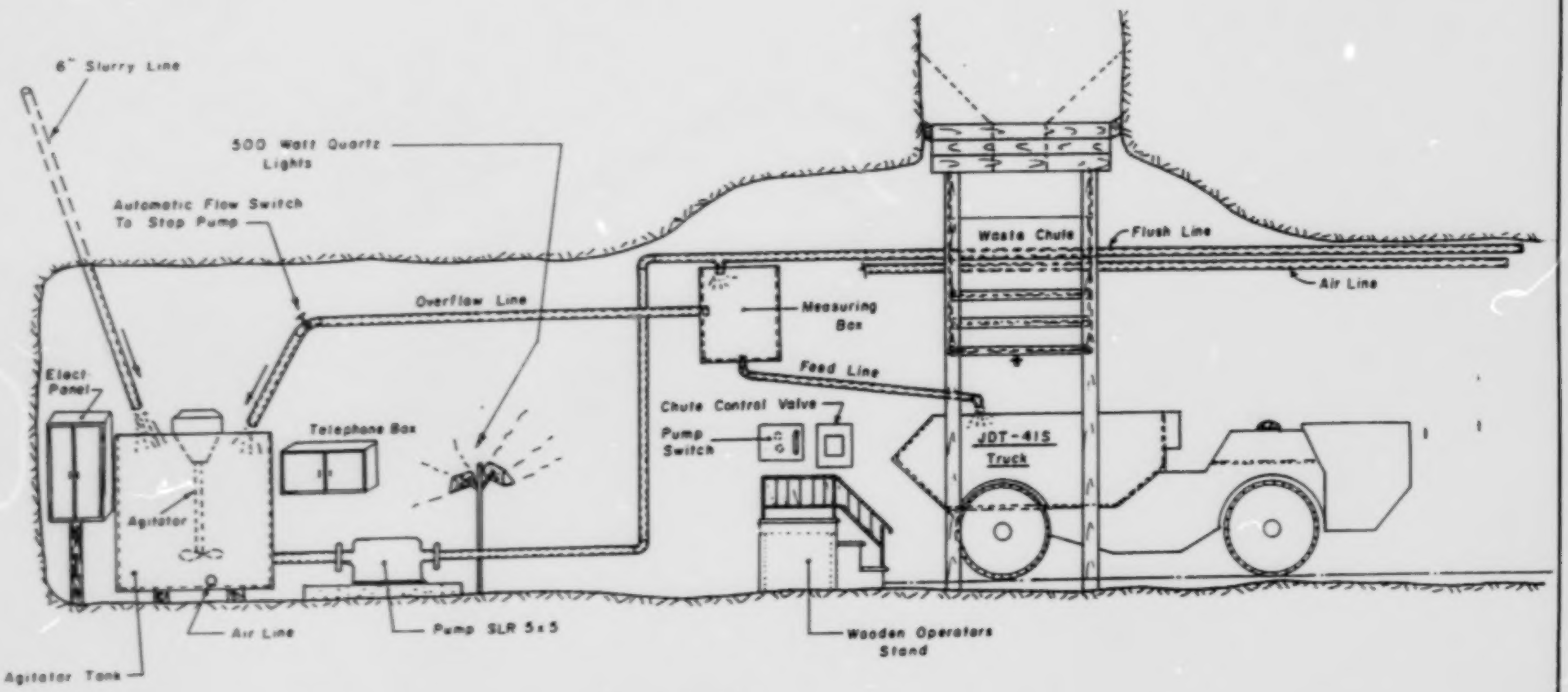
The cable fill fence is simple and easy to install.

2. Monitoring and Instrumentation:

a) In the fill plant, cement is added to the water in the mixing tank by means of a screw type feeder. A fixed time of cement feed for a required ratio of cement to water is used. The only check on this is a density check using a Marcy Scale.

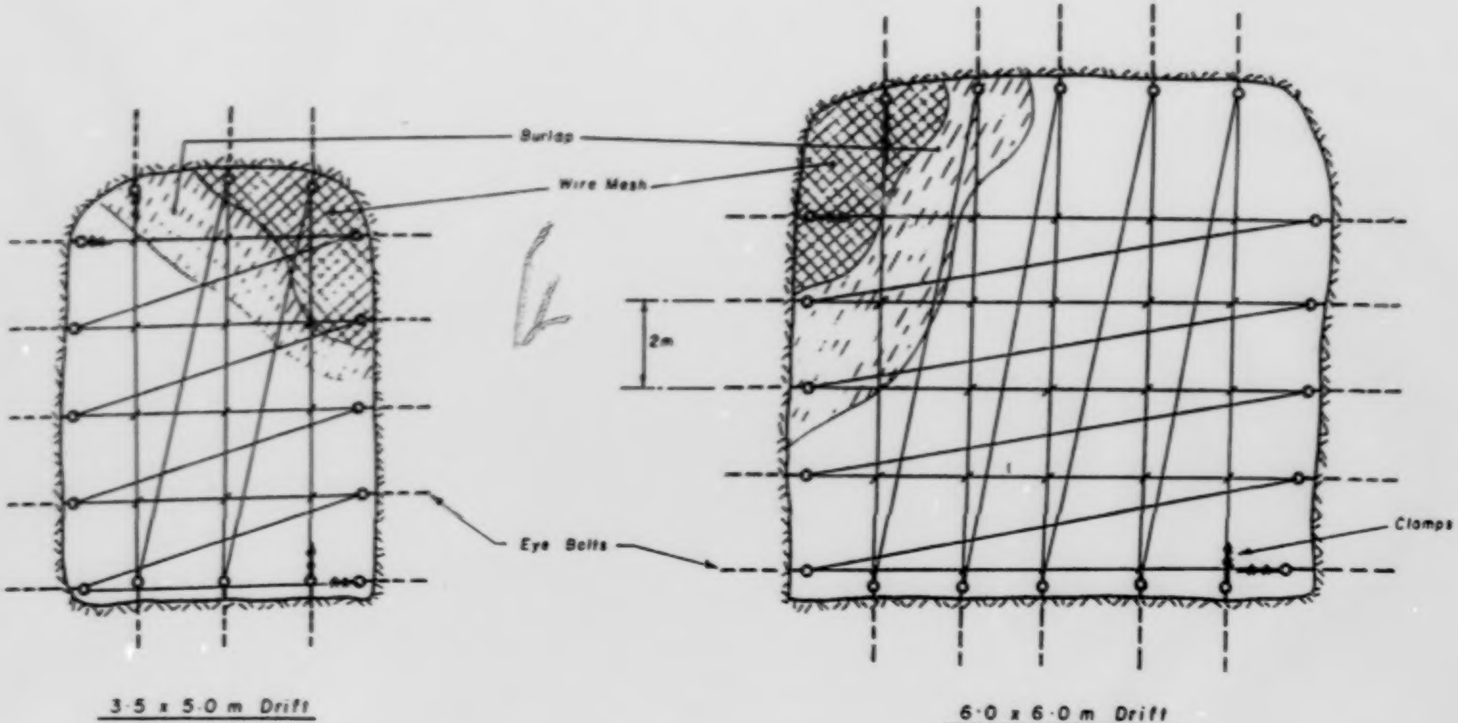
b) In the underground backfilling station, a measured amount of 55% cement-water slurry is controlled by an electric probe in a slurry measuring tank. This present volume of slurry at 55% pulp density when mixed with the 9 tonne load of waste will result in the proper ratio of cement to waste.

A further check is done by counting the number of truck loads of waste compared to the number of tonnes of slurry in each batch.


6" Slurry Line
500 Watt Quartz Lights
Automatic Flow Switch To Stop Pump
Overflow Line
Waste Chute
Flush Line
Air Line
Measuring Box
Feed Line
Chute Control Valve
Pump Switch
JDT-415 Truck
Elect. Panel
Telephone Box
Agitator
Agitator Tank
Air Line
Pump SLR 5x5
Wooden Operators Stand

LAC SHORTT

General Arrangement of Underground Backfill Station


Burlap
Wire Mesh
2m
Eye Bolts
Clamps
3·5 x 5·0 m Drift
6·0 x 6·0 m Drift

LAC SHORTT

Typical Fill Fences

BACKFILL PRACTICES

AT

LOCKERBY MINE

1.0 MINING METHODS

Lockerby Mine produces 650,000 tons of ore annually recovering Ni, Cu and Co. The mining method is blasthole stoping, both longitudinal and transverse. Stopes are 60′ x 160′ x 36′ (L.H.W.). 6½" I.T.H. drills are used drilling level to level from overcuts to undercuts.

2.0 BACKFILL DESIGN

1) Ratios and Horizons

Classified tailings fill is used at this mine in consolidated and unconsolidated form. Portland cement is the only binding agent used and sand is added to the fill in ratios of up to 20% where fill is in short supply. Tailings fill is trucked from Strathcona Mill. This dry fill is repulped in two 1800 ton mixing tanks and cement is added from a 300 ton cement silo. Sand is never used in containment plugs.

Fill to cement ratios used at Lockerby are 14:1 for containment plugs, 20:1 or straight fill for bulk mining and 10:1 or 8:1 floors.

A 14:1 containment plug is poured to a height of 25′ above the level which is well above the stope brown. Development waste is dumped into the top 2½′ of the stope and consolidated with 8:1 FTC to form a mucking floor.

Containment plugs are poured in two to three pours from the top of the open stope.

2) Dewatering Design

4" diameter drain pipes covered with a heavy cloth material are supplied in specified lengths by the manufacturer. Four of these pipes are lowered from the top of the stope and pulled through the fill fence at the access x-cut. The pipes are supported on the bottom level to prevent low points occurring.

3) Bulkhead Design

Cable fill fences are used at this mine with additional 5′ high catch fences to form a basin to retain any slimes which may pass through the main fence.

In wide areas which must be filled a cable fence with the addition of 10 ton jack posts are used. The most common area for use of this fence is in longitudinal stopes.

4) Pour Location Design

The hydraulic fill is gravity fed through 3½" and 4½" drill holes to the underground levels. Victaulic pipelines then carry the fill to the pour points.

A single pour point at the top of the blasthole stope pours a 14:1 plug behind a fill fence in the stope access x-cut. This plug is established in several small pours to about 10′ above the brown. 20:1 is then poured to top of stope with a 2½′ pour of 8:1 for a working floor. In pillar stopes unconsolidated fill is poured on top of the containment plug.

3.0 BACKFILLING COSTS

1) Cost of Fill Per Ton Placed

Ratios	T.F.	10:1	14:1	20:1	30:1
	$4.78	$14.50	$11.70	$9.70	$8.20

2) Cost of Fill Fence

The budgeted cost of fill fences at Lockerby Mine is from $7,000.00 to $10,000.00.

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

A heavier ribbed plastic pipe with a stronger drainage cloth is now used. This tower is less susceptible to damage and stretching.

Crater blasting has replaced bench blasting with a resultant lowering of dilution from 11% to 7% from fill wall sloughing.

2) Monitoring

Underground - A Geocom model 4800 vibrating wire each pressure cell is used on the fill fences. Results indicated that the highest pressure exerted at any point in the filling procedure was 2.58 P.S.I. giving a safety factor of 1:5.

Water outflow is checked and checks are made for ponded water. If problems occur filling is stopped until water is dispersed.

Extensometer and blast vibration monitors are used at this time.

A probe (developed by the U.S.B.M.) is lowered down the holes as the stope is blasted. This probe gives an accurate toping of the fill walls which will indicate any sloughing of the fill.

Computer modelling is done at this mine to determine fill strength required for any given free standing height.

5.0 PROBLEMS ENCOUNTERED

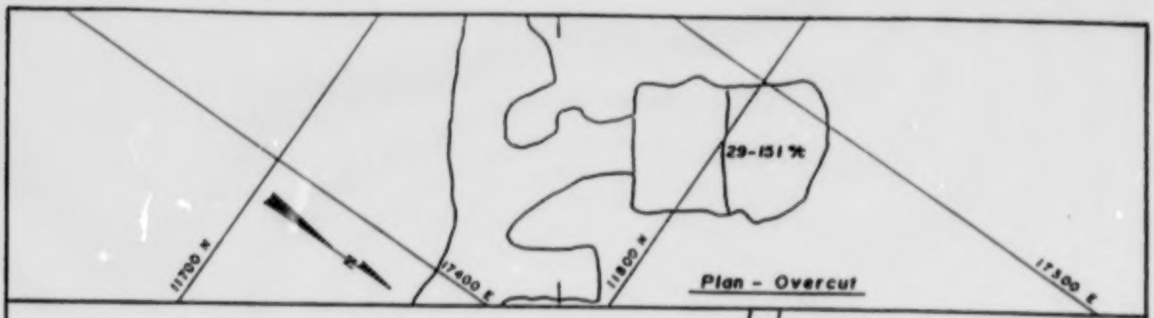
(1) Line wear.

(2) Segregation of fill.

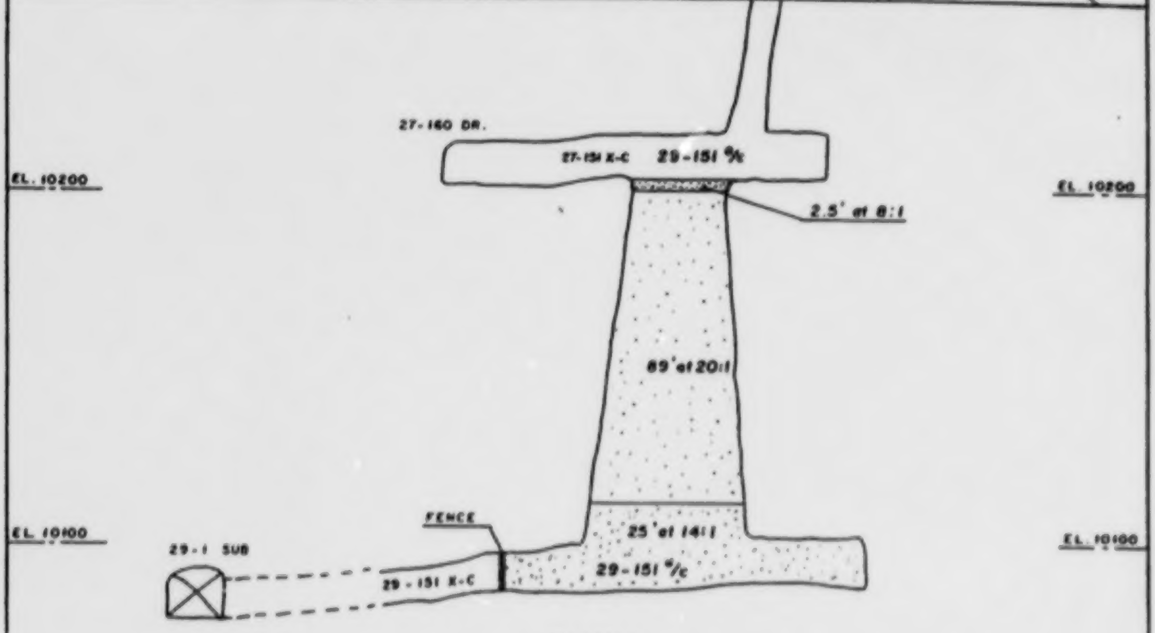
(3) Fill dilution.

(4) Higher costs due to trucking of fill.

(5) Shortage of fill.

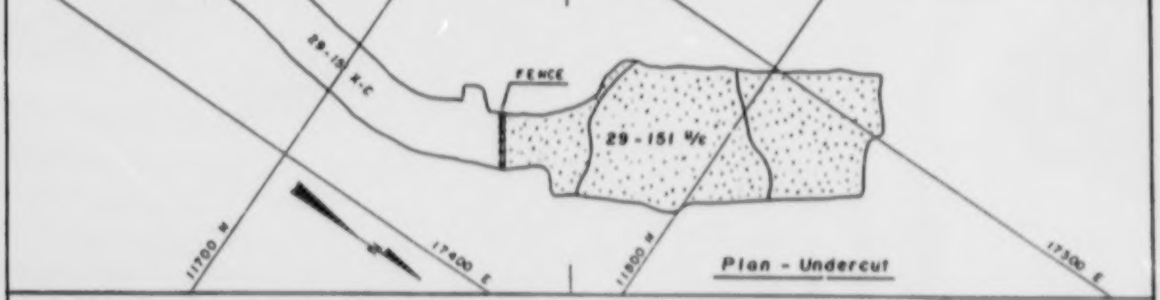

29-151 O/C
11700 N
17400 E
11800 N
17300 E
N

Plan - Overcut


27-160 DR.
27-151 X-C
29-151 O/C
EL. 10200
EL. 10200
2.5' at 8:1
89' at 20:1
EL. 10100
EL. 10100
FENCE
25' at 14:1
29-1 SUB
29-151 O/C
29-151 X-C

Section

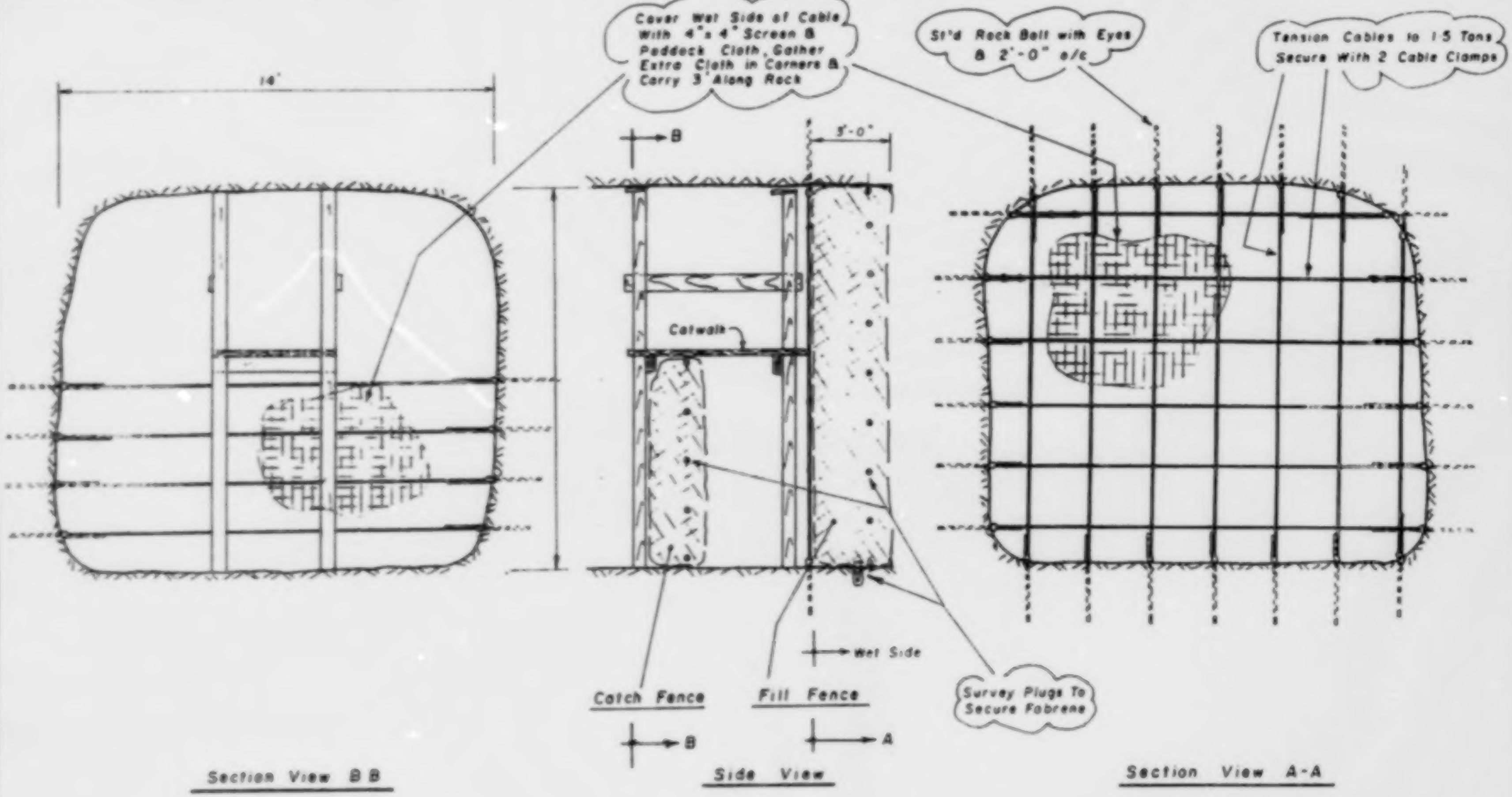
Fill Procedure	Stope Fill Calculation	Date	Fill Tons	Acc Tons	Cem. Tons	
Fill Cement Ratio	Stope Fill					
	960 + 1750 x 89 ÷ 20 = 5963					
Plug = 14:1						
Stope = 20:1						
	Tons Per Ft = 67					
Fill Procedure	O/C Floor 960 x 25 ÷ 20 = 120					
	Plug 1720 x 25 ÷ 20 = 2150					
	1720 x 13 ÷ 20 = 1118					
	Total = 3268					
	Totaltons 9351					


29-151 X-C
FENCE
29-151 O/C
11700 N
17400 E
11800 N
17300 E
N

Plan - Undercut

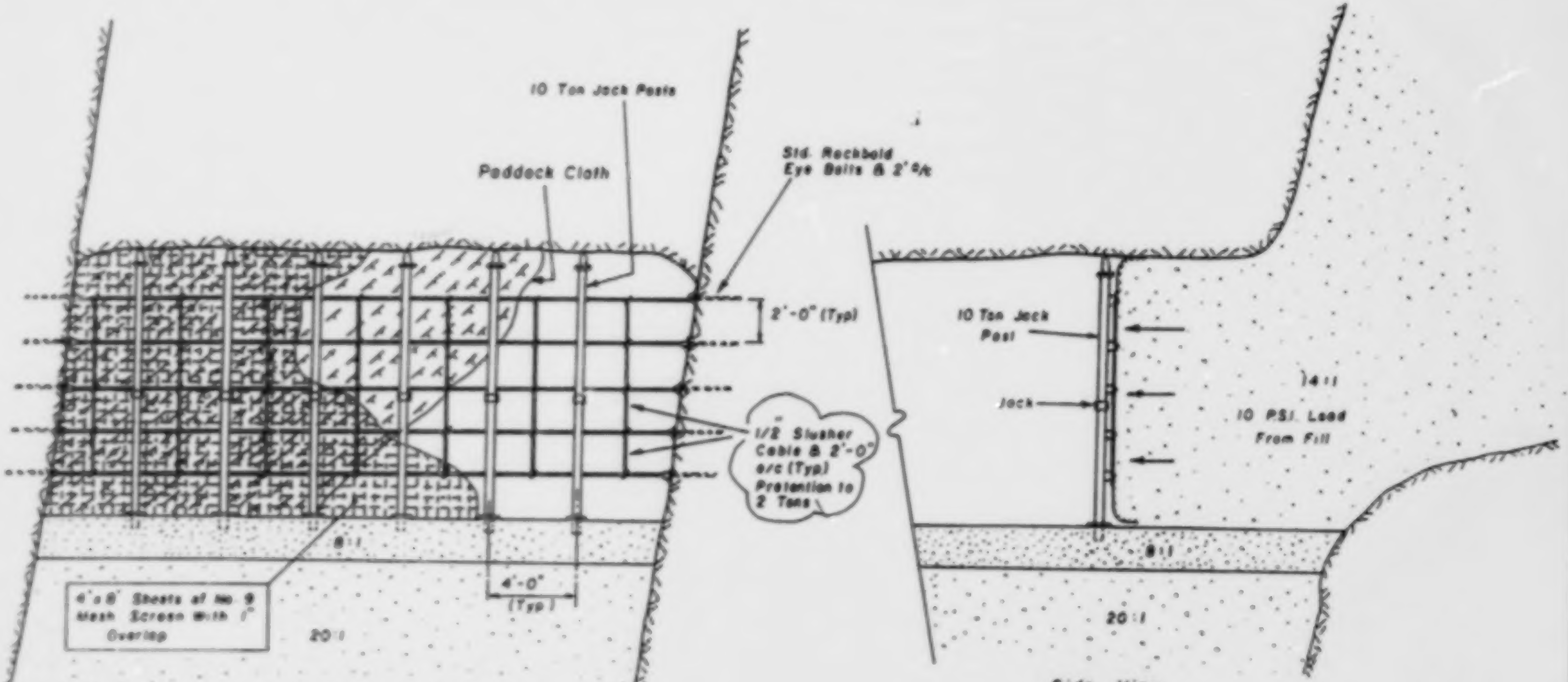
LOCKERBY MINE

Typical Blasthole Stope - Fill Procedure


Cover Wet Side of Cable With 4"x 4" Screen & Paddock Cloth, Gather Extra Cloth in Corners & Carry 3' Along Rock
St'd Rock Bolt with Eyes @ 2'-0" o/c
Tension Cables to 1.5 Tons Secure With 2 Cable Clamps
14'
B
5'-0"
Catwalk
Wet Side
Catch Fence
Fill Fence
Survey Plugs To Secure Fabrene
B
A
Section View B B
Side View
Section View A-A

Cable — Fill and Fill Fences

FALCONBRIDGE - LOCKERBY MINE


10 Ton Jack Posts
Paddock Cloth
Std. Rockbolt Eye Bolts & 2' o/c
2'-0" (Typ)
1/2" Slusher Cable & 2'-0" o/c (Typ) Pretention to 2 Tons
4'-0" (Typ)
8:1
4'x8' Sheets of No. 9 Mesh Screen With 1" Overlap
20:1
10 Ton Jack Post
Jack
14:1
10 PSI. Load From Fill
8:1
20:1

Front View

Side View

St'd Cable Fill Fence — in Wide Areas

Falconbridge — Lockerby Mine

BACKFILL PRACTICES

AT

ONAPING MINE

1.0 MINING METHODS

Onaping Mine is producing 365,000 tons of ore annually recovering 1.93% Ni and 0.40% Cu. The predominant mining method at present is mechanized cut-and-fill, leaving non-recoverable post pillars. Drilling is done with McKee jumbos and one and two boom Atlas Copco electric hydraulic jumbos. Mucking is with two and five yard electric and diesel scooptrams.

2.0 BACKFILL DESIGN

1. Ratios and Horizons

Classified tailing is trucked from Strathcona Mill in filter cake form. A high capacity beltavator has been recently installed which transfers the tailings into an 800 ton settled storage tank and from there into the system where additional water and binders can be introduced into the fill to obtain desired density and ratio. Sand may also be added at the mixing tank location as required (see attached schematic drawing).

When consolidated fill is required, a 32:1 fill is used with binding agents of 50% slag and 50% Portland cement. Straight fill is poured in the cut-and-fill stopes where no adjacent mining will take place and at present special fill floors are not poured. Sand is added to this fill but the ratio is not closely monitored.

In horizontal sills, 12:1 is used where width is less than 40 feet and 8:1 where width is over 40 feet. Screen 4" x 4" is laid prior to pouring this fill. The screen is laid across the floor, up the walls, and bolted.

2. Dewatering Design

a) Sill areas: 12' x 12' x 6' drain sets are constructed on the sill and are located by the surveyors. Three or four inch diamond drill holes will be drilled through rock into these drain sets to draw off the collected water.

b) In the normal cut-and-fill areas, a cable fence is constructed as per attached drawing and 6 x 6 cribbed manways are covered with fabrene to facilitate drainage.

3. Bulkhead and Fill Fence

There are no bulkheads installed. See attached drawing of cable fill fence.

4. Pouring Design

There is no standard pouring design for the cut-and-fill stopes.

3.0 BACKFILL COSTS

1. Cost per ton of fill mined:

Ratios -	SF	8:1	10:1	16:1	30:1	40:1
	$5.70	$17.50	$15.10	$11.60	$8.80	$8.10

2. Fill fence cost - not available.

4.0 INNOVATIVE BACKFILLING PRACTICES

1. Methods

- Screen only on horizontal sill before the fill is poured.
- Beltavator conveyor for rapid handling of caked fill.
- Horizontal sill at Craig is established with 8′ of waste then consolidated with 8:1 C.T.F.

2. Monitoring

An ultrasonic high level detector in the truck acceptance hopper stops the feeder to No. 1 settler tank and closes the truck receiving door.

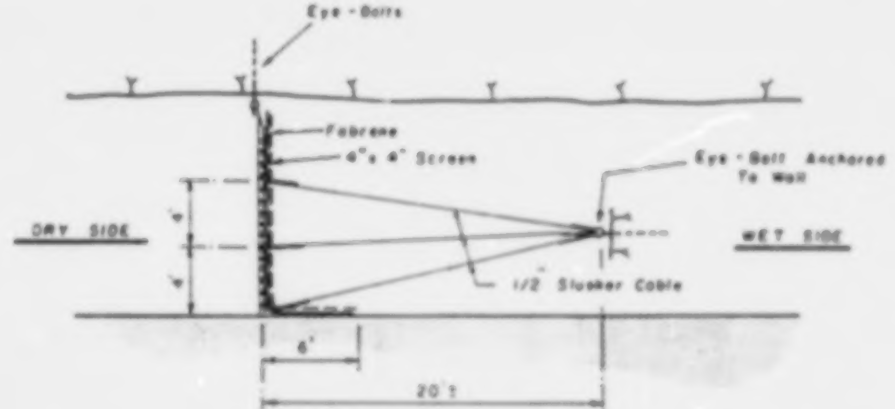
An automatic density surge checks the density of the fill prior to entering the underground fill holes.

5.0 PROBLEMS ENCOUNTERED

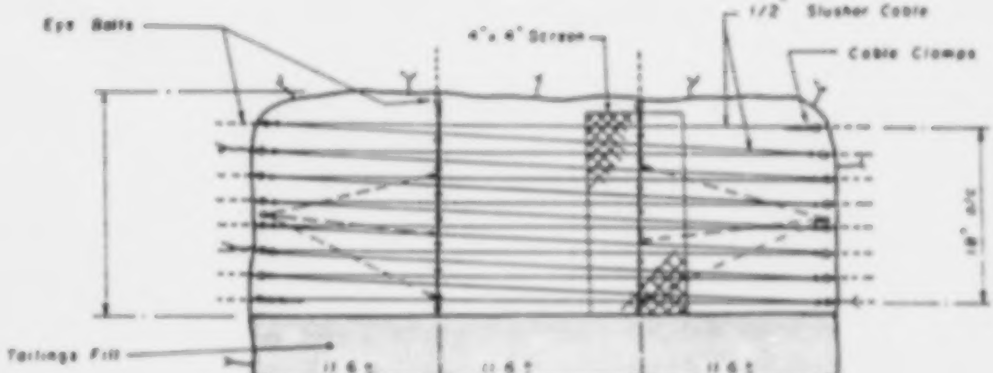
- Too many coarse particles cause pipe wear and valve malfunctions.
- Some segregation.
- No monitoring of sand additions.

CABLE FILL FENCE INSTALLATION NOTES

1 - Drill 2' Deep Holes for Eye Bolts in Wall and Back
2 - Install Eye Bolts and Tighten
3 - String Slusher Cable Between Wall Eye Bolts
4 - Starting at Bottom of Fence Tighten Cable Between Eye Bolts with Come-Along and Clamp Cables Together at Eye Bolts
5 - String Loops of Slusher Cable From Roof Eye Bolts Around Both Sides of Cable Fence. Clamp Loop Together at 18" o/c.
6 - Place 4"x 4" Screen Vertically Along Wet Side of Cable Fence and Extend 6' Along Floor. Overlap Screens By 4" on Edges and Bind Together and To Cables With Wires.
7 - Cover Screen With Fabrene (Extend Screen and Fabrene 3' Along Walls as a Seal)
8 - Attach 3 Lengths of Slusher Cable to Vertical Cables at 4' o/c. Connect to Eye-Bolts Located in Stope Wall Approx. 20' Behind Fence.


Eye-Bolts
Fabrene
4"x 4" Screen
Eye-Bolt Anchored To Wall
DRY SIDE
WET SIDE
4'
4'
1/2" Slusher Cable
6'
20'±

X - Section View


Eye Bolts
4"x 4" Screen
1/2" Slusher Cable
Cable Clamps
18" o/c
Tailings Fill
11 6 ±
11 6 ±
11 6 ±
35' Typ ±

Elevation View

Falconbridge — Onaping Mine

Cable Fill Fence in Cut and Fill Stope

BACKFILL PRACTICES

AT

PAGE-WILLIAMS MINE

1.0 MINING METHODS

Lac Mineral's Page Williams Mine is scheduled for a production rate of one million tons of ore annually. The principle mining method is open stope blasthole, drilling 4 1/2" downholes and blasting to a Robbins slot raise. Sub-levels are at 25 m intervals. Stopes average 40m x 20m x 25m (LxWxH). There are 350 employees.

2.0 BACKFILL DESIGN

1) Fill Ratios - The only fill used at this mine is C.R.F. with cement as the binder in ratios of 4 to 7% by weight. Flyash will replace a portion of the cement when a new silo is installed.

   Surface waste from the open pit is crushed to minus 6" resulting in a ratio of 75% +3/8 and 25% -3/8". This aggregate is stacked and picked up by a front-end loader as required and dumped into backfill raises. At the first underground level the fill may be passed to the upper zone or conveyed by permanent conveyor to raises which will pass it to the lower zone. Freestanding walls have stood up to 50m with a dilution rate of 10%.

2) Dewatering Design and Bulkheads - A standard fill fence (see dwg. L400-10-04) of a more sophisticated than usual design provides excellent and clean drainage of the water. Where ground conditions have deteriorated so as to incur a safety hazard, self forming bulkheads are poured using a 7% cement to aggregate ratio. Fences have drain pipes and fabrene for water control.

3) Pour Location and Design - There are presently three areas being filled at one time. One area is filled by portable conveyors and the other two are using scooptrams which will in the future be replaced by fill trucks. The choice of these two methods is based on an economics decision. Normally 40,000 tonnes of fill or less will result in using the scoop haulage method. Fill production is 4,000 tons capability per day.

4) Filling Using Scoops - A conveyor loads directly into the bucket and a metered amount of binder slurry is piped into the aggregate as it falls into the bucket. The scoop then travels to the dump site which is usually a Robbins raise or it can be dumped directly into an open stope. It should be noted that the Robbins raises are drilled through several sub-levels in one lift and are used as both slot raises and fill raises.

5) Filling Using Portable Conveyors - Portable conveyors carry the aggregate to the borehole or to the open stope. Baffled chutes are often used at the head end of the conveyor to further mix the binder with the aggregate. A spray of binding slurry falls on the aggregate as it leaves the head end of the conveyor.

3.0 BACKFILL COSTS (Overall cost of fill placed is $10.00/ton)

4.0 INNOVATIVE BACKFILLING PRACTICES

1) Methods

i) Using high ratio (7% cement with some sand addition) to form bulkheads in areas of hazardous ground conditions without installing a fence.

ii) Advanced instrumentation underground for better control of cement addition.

iii) Combining slot raises and fill raises.

2) Monitoring and Instrumentation

This mine does not have a lab on site but tests done by Canada Cement indicate a uniaxial compressive strength of 100-200 p.s.i. Screen analysis carried out indicate the relationship between unixial compressive strength and percentage of fines in the aggregate. A content of 25% (minues 3/8") results in the highest compressive strength.

The fill plant is controlled by the latest in instrumentation and monitoring devices and in addition to this the underground backfilling stations are fully instrumented. One instrument on the underground operators control panel receives information from the weightometer on the aggregate conveyor and calculates the amount of cement slurry required for the preset ratio. It then sends a message to the flowmeter on the slurry line and controls the flow of slurry. The extensive instrumentation leads to a highly accurate cement to fill ratio but it should be noted that careful and constant calibration of the instrumentation is required.

Page Williams Mine is planning to build an underground testing lab.

Dupont has established a vibration curve for blasting at various distances from standing fill at this mine.

5.0 PROBLEMS ENCOUNTERED

i) Some segregation particularly in large areas.

ii) Segregation at stacked stockpile of aggregate.

iii) High fines content at lower levels due to attrition in passes.

iv) Freezing of aggregates in stockpile in winter causing hangups in fill raises.

CHAPTER 4

CONVENTIONAL HYDRAULIC BACKFILL, AN OVERVIEW

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  4.1.2 Fill Materials

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  4.2.2 Segregation
    4.2.2.1 Effect of Discharge Point Location
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        - 4.4.2.5 Consolidated Fill Drainage
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- 4.5 Fill Containment Barricades
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        - 4.5.2.1 Containment Plugs
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        - 4.5.3.2 Drainage Blankets
        - 4.5.3.3 USBM plug
    - 4.5.4 Discussion
    - 4.5.5 Fill Fence Performance and Water Pressure Monitoring

- 4.6 Preparation

- 4.7 Research and Development

4.1 INTRODUCTION

Hydraulic fills are those which use water as a medium of transport. A conventional hydraulic fill is one which is transported at a concentration of 50% up to about 72% solids by weight (40% to 50% by volume). Hydraulic fills at densities higher than this are defined as paste or high density hydraulic fills in this report. These high density fills exhibit different flow characteristics, require different preparation and are not yet widely used. Conventional hydraulic backfill is the most versatile and widely used fill type.

This chapter is presented as background information on the theory and practice of conventional hydraulic filling. It attempts to identify the strengths, weaknesses and compromises inherent in placing such fills. The limitations of hydraulic fill have prompted the search for, and development of innovative fill practices such as paste filling. The strength and stiffness limitations of hydraulic fill have prompted the increasing application of rock fills. The review presented here is only brief.

An important limitation in the use of hydraulic fill is the conflict in engineering a fill to have adequate transport characteristics and at the same time trying to achieve optimum in place performance. The properties which allow ease of transport often adversely affect its strength properties. Engineering a hydraulic fill is not an exact process and often a frustrating one, trying to compromise opposing requirements.

4.1.1 Liquefaction

Liquefaction is a dangerous condition which can occur in unconsolidated fills due to vibratory loading from blasting or seismic activity. Liquified fill acts as a high density liquid capable of exerting large hydraulic pressures against containment structures. If these fail, fill will flow into the mine openings with severe safety implications.

The risk of liquefaction is the primary consideration in engineering a hydraulic fill system which is to be placed unconsolidated.

Instantaneous pore-water pressures in saturated granular materials, such as undrained hydraulic backfill, when subjected to cyclical or pulsating loads may result in the mass of material having zero shear strength, the definition of a fluid (Ford 1978). Consider the Mohr Coulomb equation in its most general form, which includes consideration of pore water pressure.

$$S = C + (P - u)\tan\varnothing$$

where;
S = shear strength, kPa
C = cohesion, zero for unconsolidated fill, kPa
P = normal stress, kPa
u = pore water pressure, kPa
$\varnothing$ = angle of internal friction, degrees

If cohesion is zero and a condition exists such that pore pressure equals the normal stress then the fill will have no shear strength and may be considered as a fluid. As an example, it has been stated that explosive charge weights of 90 kg to 227 kg per delay will cause liquefaction of saturated unconsolidated tailings backfill, when detonated within a distance of 15 m from the fill. If the material has a high percolation rate so that the backfill is kept in an unsaturated state, liquefaction of the fill will not occur.

Liquefaction is not possible with consolidated fill which has cured sufficiently to develop even small amounts of true internal cohesion.

4.1.2 Fill Materials

Mill tailings are the most common hydraulic fill material; numerous surveys of mining operations has confirmed this. They are cheap and available and so, are usually the material of first choice. In addition their use as fill helps to alleviate a surface disposal problem.

Mill tailings are not always available; e.g. too fine to be used (some gold mining operations), contain high residual metal values (future retreatment potential), or the ore is milled at a remote location. In these cases quarried alluvial sand is often used as fill. Alternatively, some mines have employed granulated slag or gravels

4.2 IN PLACE PROPERTIES

The performance of a hydraulic fill depends not only upon the inherent material properties such as particle size and uniformity, solids density, and the amount and type of binder, but also on the conditions of transport and placement.

4.2.1 Strength

The strength of a hydraulic fill will commonly be expressed in terms of unconfined compressive strength, UCS. Shear strength is a more logical description of fill strength, but UCS is quick and easy to measure, and may be considered a direct reflection of shear strength (failure under unconfined compression is a shear failure). Strength figures discussed in this chapter generally refer to UCS unless otherwise stated.

Some design procedures for determining what fill strength is required in different applications were given in chapter 2. The two principal variables which the engineer has practical control over in the designing the strength of a hydraulic fill are, the amount of binder added and the size distribution of the fill materials. The conditions of placement (drainage facilities provided, pour point location etc) will also play a lesser, but still important role.

In practice control of the particle size distribution is severely constrained; permeability must be maintained to prevent liquefaction, and economic considerations require the use of the cheapest materials available, which may not have the ideal sizing. Hence control of the amount of binder addition becomes the one variable which can be easily managed in order to produce a backfill of the required strength. However, it is also generally the highest cost item in backfill, so its use must be judicially regulated.

The relationship between binder content, particle size and UCS, as proposed in design equations by Swan (1985) and by Mitchell (1982) were discussed in chapter 2. Some binder contents actually used at mines in Ontario can be found in chapter 3. Typical binder addition rates used by Falconbridge, Sudbury Operations for various applications with tailings fill are given below (ratio tailings:binder by weight). These indicate the sorts of ratios being used in the industry, which may not necessarily match what the design equations suggest. Practical experience and the degree of acceptable risk are often better guides to determining what ratios to use than the equations.

1. Blasthole or cut and fill stope. Fill required to stand unsupported during pillar extraction. Ratio dependent upon stope height and width; 12:1 to 32:1 for exposures up to 70 m high. A high

value pillar would encourage the operator to take less risk and use a higher binder content in the fill. At least one month is allowed for curing.

2. Blasthole stope plug. Fill required to act as containment barricade. Ratio dependent upon stope height and geometry; 8:1 to 16:1. Must have good strength at 7 days.

3. Sill mat for recovery of pillar from underneath or in undercut and fill mining. Ratio of 8:1 or better. Very long curing times available, but high ultimate strength required.

4. High strength stope floor in cut and fill. Ratio of 8:1 to 10:1. Must be able to attain good strength at an early age to allow mining of the next cut to proceed.

Table 4.1 below relates the laboratory determined strength of Falconbridge tailings fill at various curing times to the binder addition rates. This shows the typical improvement in strength with curing time and the sorts of strengths required for different applications.

Tailings:Binder	Unconfined Compressive Strength, kPa		
	7 days	14 days	28 days
8:1	889	1020	1331
12:1	455	552	690
16:1	324	365	455
24:1	152	172	269
32:1	110	138	193

Table 4.1, Laboratory measured strength of Falconbridge consolidated hydraulic tailings fill. (68% solids concentration by weight, Type 10 portland cement binder)

The relationship between mining schedule, which will determine the curing time available in a particular stope, and the amount of binder which must be added to achieve the desired strength is obvious. Short curing times require higher binder contents, which means higher backfilling costs. Significant savings in binder cost are possible by scheduling operations to provide longer curing times.

4.2.1.1 Strength and Solids Concentration of the Slurry

The solids concentration of a hydraulic fill slurry will have a major impact on the strength of the fill.

Porosity of an in place fill decreases with increasing solids concentration of the slurry, e.g higher the fill pulp density, the lower the porosity. From previous discussions we expect a higher strength fill due to greater inter particle friction as the porosity decreases. Figure 4.1 illustrates the difference in strength obtained in laboratory samples of Falconbridge tailings fill at solids concentrations of 68% and 70% by weight. The difference is significant.

The effect of solids concentration on strength is greatest for well graded fills; at low concentrations the finest particles are kept in turbulent suspension while the large particles settle out forming a loose open structure (as for a uniformly graded fill). The finest particles will tend be washed out with the drainage water or settle in discrete layers. Fines loss with the drainage water may be 20% or more (Clark 1988). This is of particular interest when a binder is added to the fill. At low solids concentration a higher loss of the fine binder particles with the drainage water can be expected. Hydraulic fill pulp density should be kept as high as possible.

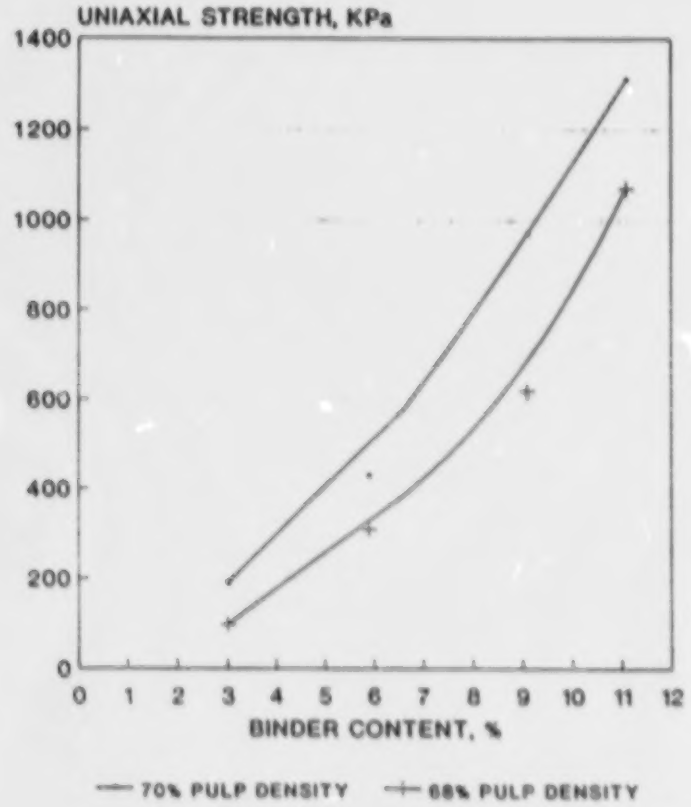
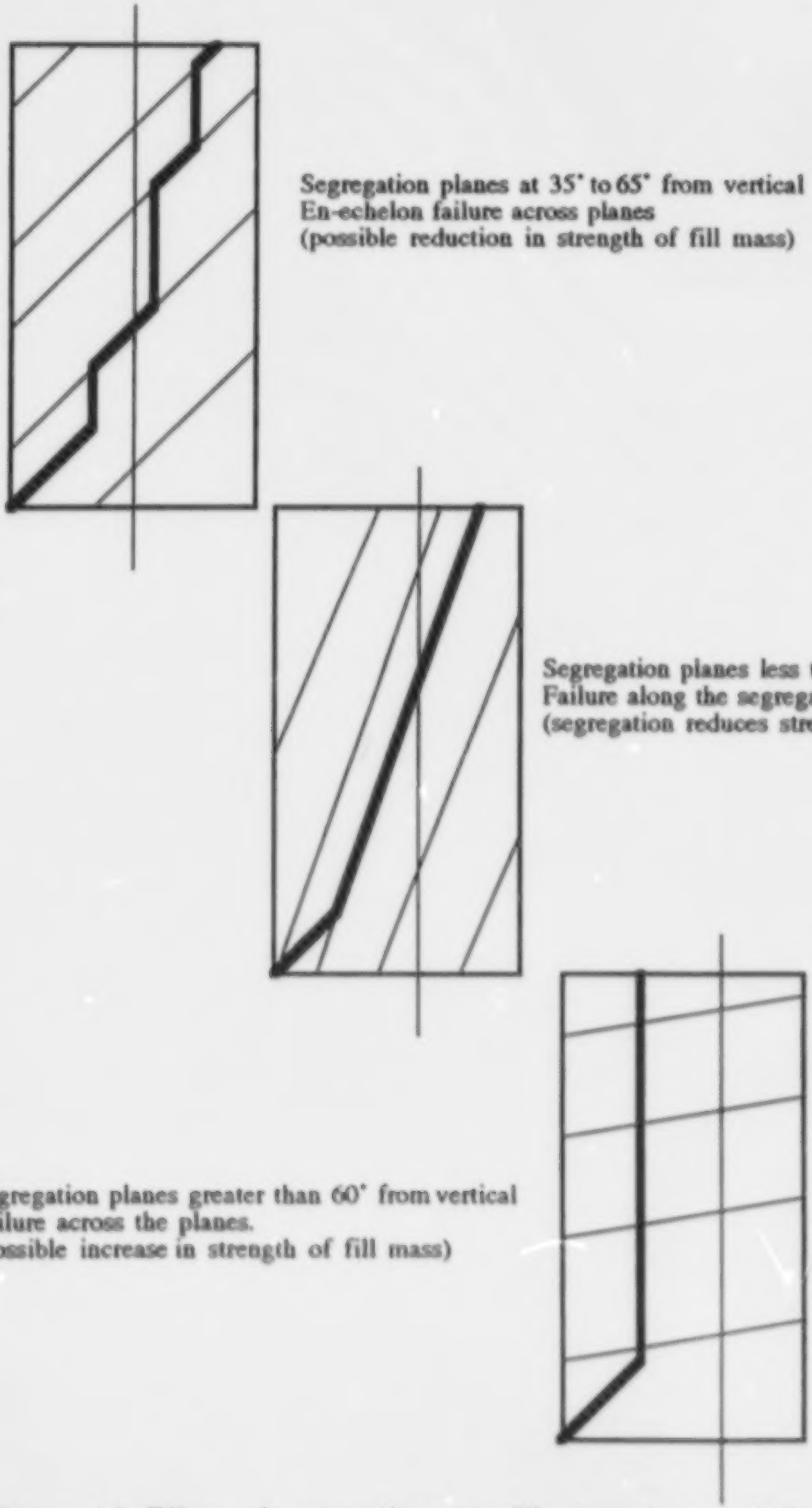


Figure 4.1 **Strength of Falconbridge hydraulic tailings fill poured at 68% and 70% solids. (Normal portland cement as a binder).**



Segregation planes at 35° to 65° from vertical
En-echelon failure across planes
(possible reduction in strength of fill mass)

Segregation planes less than 35° from vertical
Failure along the segregation planes
(segregation reduces strength of fill mass)

Segregation planes greater than 60° from vertical
Failure across the planes.
(possible increase in strength of fill mass)

Figure 4.2, Effect of segregation on fill strength and failure planes. (after Barrett, 1973)

4.2.2 Segregation

Segregation is a phenomenon common to all backfills. It is the spatial distribution of varying particle sizes in a fill mass. It causes the fill mass to be non homogeneous and may result in an overall reduction in fill mass strength by the creation of planes of weakness. Paste and high density hydraulic fills were developed partly to minimize the problems associated with segregation.

When backfill is being poured hydraulically into a stope, particles will become segregated by size. A beach will form sloping away from the pour point at a shallow angle. Three zones of fill with varying average particle sizes are formed in this beach structure (Jerabek & Hartman 1965). The first zone is located in the area nearest the pour point, and consists of particle sizes representative of the overall size distribution in the backfill mix. The next zone, located further from the pour point down the beach, contains mostly coarse particles. The third and outermost zone contains a predominance of fines. This segregation is a result of differential settling rates between particles of different size.

Binders added to hydraulic fill have a comparatively fine particle size compared to the tailings or sand and this will often result in non uniform distribution of binder through the fill mass due to segregation effects. Zones with low binder content may reduce the overall mass strength of a backfill. Segregation may cause in place fill strength to be very different to the assumptions made based upon laboratory testing.

Segregation will also occur in a vertical sense due to differential settling of particles in pools of excess drainage water. This is the characteristic banding found in fill exposures. The banding will be more definite at the stope edges compared to the center.

When filling with cemented hydraulic backfill, strength is influenced by the orientation of segregation planes. As an example, Barrett (1973) conducted laboratory strength tests on samples in which a segregation rill was artificially created. The samples were subjected to undrained triaxial compression tests with the angle between the principal stress axis and the plane of segregation being varied. The following held true:

1. When the angle was greater than 60 degrees, failure occurred across the layering and net strength was equivalent to that of unsegregated samples;

2. When the angle was between 35 and 60 degrees, the failure was en-echelon type and net cohesion was reduced;

3. When the angle was less than 35 degrees (i.e. the angle of friction), failure occurred along the layers with a further reduction in net cohesion.

Figure 4.2 illustrates the ideas.

4.2.2.1 Effect of discharge point location

The location of the discharge points within a stope will control the beach formation and thus the orientation of the planes of segregation in a stope. It is necessary to locate discharge points so that the formation of weaker zones that dip in the direction of faces which are to be exposed is avoided. A weak zone would be those areas which are deficient in cement by comparison with the bulk pour ratio. Consider the stope geometries and discharge locations shown in figure 4.3, from which some conclusions on discharge point geometry can be made.

1. A central discharge point is best for minimizimg the potential problems associated with segregation.

2. A vertical walled stope will exhibit more favorable segregation affects. To achieve the same consistency with a sloped wall stope, the discharge point should be moved during the pour to keep the impact point as close to the center of the stope as possible. Falconbridge Limited, Sudbury Operations have used a "clothesline" arrangement to achieve this. The fill line is strung from a cable previously run across the back of the stope. The clothesline allows the discharge to be moved to any point over the stope cross section.

3. The pouring of fill directly onto the footwall of inclined stopes should be avoided because of potential for damage to the drainage pipes.

4. The same segregation effects shown in section in figure 4.3 will repeat themselves in the third dimension should the stope have any appreciable length. A second pour point should be considered for large stopes where segregation could be unfavorable.

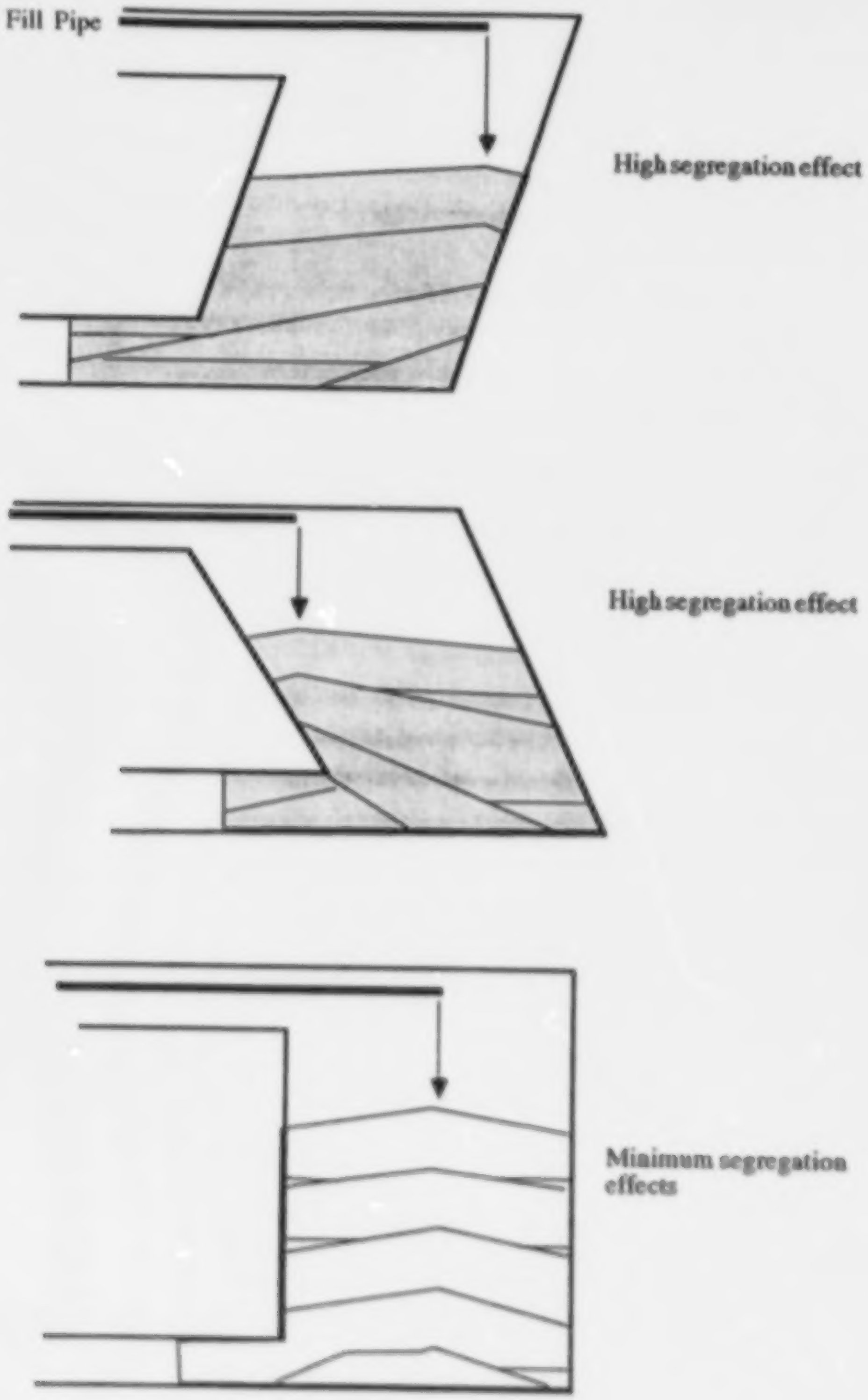

Fill Pipe
High segregation effect
High segregation effect
Minimum segregation effects

Figure 4.3, Effect of discharge point on segregation in large stopes. (exaggerated segregation)

4.2.3 Measured Fill Properties

The important engineering properties of placed backfill are;

1. Unconfined Compressive Strength, UCS, kPa
2. Friction Angle, Ø
3. Cohesion, C, kPa
4. Elastic Modulus, E, GPa
5. Poissons ratio, GPa
6. Moisture Content, % (weight water as % of wet weight of material)
7. Porosity (%) or void ratio
8. Unit Weight, kN/m^3
9. Dry Bulk Density, Kg/m^3
10. Solids Density, Kg/m^3
11. Percolation Rate, mm/hour

The important properties can all be measured in the laboratory relatively easily (see appendix H). However their measurement in place is quite difficult and not always representative because of non uniformity due to segregation effects. Work by Falconbridge, partly under the auspices of a previous CANMET sponsored project - In Situ Monitoring and Computer Modeling of a Cemented Sill Mat and Confines During a Tertiary Stage Pillar Recovery, has measured some of these properties in place. Because of the variability of fill due segregation the results are presented divided into only two types of fill, weakly and strongly consolidated.

Property	Weakly Consolidated (tailing:binder≥24)	Strongly Consolidated (tailing:binder<10)
UCS, kPa	200 - 1000	2000 - 4000
Ø	35° - 50°	no reliable results
C, kPa	100 - 400	1000 - 1500
E, MPa	50 - 100	300 - 500
Moisture content	17%	15%
Unit weight, kN/m³	20	18.5
Dry bulk density, kg/m³	2100	2100
Porosity	N/A	N/A
Solids density, kg/m³	3000	3000

Table 4.2, In place properties of Falconbridge consolidated hydraulic fill. (Type 10 normal portland cement as a binder)

For comparison, table 4.3 below gives average values of the same properties as measured in the laboratory. These values are for hydraulic fill at 68% solids and for the size distribution particular to Falconbridge's Strathcona mill classified tailings.

	Tailing:Binder			
Property (at 28 days)	8:1	16:1	24:1	32:1
UCS, kPa	1331	455	269	193
Ø	36°	36°	36°	36°
C, kPa	740	190	140	70
E, MPa	350 - 380	80 - 120	40 - 60	40 - 60
Moisture content	18%	18%	18%	18%
Unit weight, kN/m³	21	21	21	21
Dry bulk density, kg/m³	1800	1800	1800	1800
Porosity	42%	42%	42%	42%
Solids density, kg/m³	3070	3070	3070	3070

Table 4.3, Laboratory measured values of Falconbridge tailings fill properties. (Type 10 normal portland cement as a binder)

4.2.4 Permeability

The danger which is posed by liquefaction has been stated. The one sure way to eliminate the danger is to add binder to give fill some cohesion. However when not required for strength development the addition of binder is an expensive solution. In most cases the fill will be size classified (fine particle sizes removed) to ensure that it will be sufficiently permeable to ensure it remains well drained. Classification removes much of the finest particle size fractions known as slimes from the fill. Slimes will tend to block the drainage paths in the fill mass, i.e reduced porosity.

As a guide in initial design of a backfill system, to ensure adequate permeability, a couple of generalizations are;

1. A suitable material for backfill should be limited to a maximum content of 20% minus 45 microns in diameter.
2. Fill size distribution should have a uniformity coefficient, C_u of between 4 and 6.

A percolation rate of 100mm per hour is the accepted industry standard for hydraulic fill permeability. This is not based upon much scientific foundation and many operations place fill with rates significantly below this standard with out problem. However it is a good standard for which to aim in initial design. Once in operation the percolation rate and fill sizing can be adjusted on the basis of practical experience.

The classification of tailings is a compromise between recovery of material for use as fill and its drainage characteristics. Removal of too much of the fine size material will limit the quantity of tailings available for use as fill. The classification should be designed to give maximum recovery with acceptable drainage performance. Producing enough fill material to meet production demands is a problem at many mining operations.

The classification of fill to remove the fine fractions is the great contradiction in hydraulic fill design. By classifying a more uniformly sized material is created, such a material will exhibit higher porosity and generally lower strength. Stronger fill could be placed if the fine particles were left as part of the fill. In addition fill slurries will usually flow better with higher fines content; fines tend to support coarse particles in suspension, effectively decreasing the critical velocity.

4.2.4.1 Permeability of Consolidated Fill

The addition of a binder to fill will significantly reduce the fill permeability. Fine binder particles act as slimes and block the drainage paths in the fill mass. As the cementitious reactions proceed the formation of interstitial cement gel further acts to block drainage paths and reduce permeability. The reduction in permeability increases with the amount of binder added and the fineness of the fill material. The pulp density of the pour and size distribution of the fill will also influence how much reduction in permeability a binder will cause.

The long term permeability of a consolidated fill is not a concern because cohesion will prevent liquefaction. However the short term effects of binder on fill drainage during and just after pouring may cause problems. The reduction in permeability of a consolidated fill during the pour will often result in the need to rely more upon decant drainage than percolation drainage (e.g. better drainage facilities are required).

4.3 PLACEMENT AND TRANSPORT

The delivery of backfill to underground levels and its distribution to individual stopes is generally accomplished through a network of holes and pipes extending from surface to the stopes to be filled. The basic design principals for a hydraulic fill distribution system are as follows.

1. Solids concentration of the slurry should be as high as possible in order to minimize the volume of drainage water , maximize the placement rate, and minimize the in place fill porosity.

2. Operating line flow velocity should be as low as possible in order to minimize wear and pressure in the delivery line, but must be high enough to hold the coarse particles in suspension during transportation.

3. Flow velocity should be constant throughout the delivery system to minimize surging, vibration and line plugging.

4.3.1 Solids Concentration

Hydraulic fill is commonly described by its solids concentration in terms of weight percent (easiest to measure). A conventional hydraulic fill will generally contain 50% to 72% solids by weight depending upon the specific gravity of the solids.

The flow characteristics of a fill are more directly related to its volumetric concentration. For a conventional hydraulic fill it has been determined by experience that the maximum flowable concentration for a classified mill tailings fill slurry is around 40 to 50% solids by volume. Concentrations over 50% are better described as high density hydraulic or paste fills. These are typical values only.

Solids concentration can be expressed either by weight, C_w or by volume, C_v.

$$Cv = (\text{Volume of Solids}) / (\text{Volume of Solids} + \text{Water})$$

$$Cw = (\text{Weight of Solids}) / (\text{Weight of Solids} + \text{Water})$$

$$Dm/Ds = Cv/Cw$$
$$Dm = CvDs + (1-Cv)Dl$$
$$Cw = (CvDs)/(CvDs + (1-Cv)Dl)$$

where:
Cv = Solids concentration by volume %
Cw = Solids concentration by weight %
Dm = slurry specific density
Ds = solids specific density
Dl = liquid specific density, unity for water

Slurry solids concentration is kept high for the following reasons;

1. Maximum fill delivery rate with minimum drainage water to handle.

2. Lower water solids ratio gives lower porosity and greater strength fill mass.

3. Less drainage water results in less fines and binder loss from the fill mass.

4. Water and entrained air affect the performance of the binder in consolidated fill. In concrete the strength declines rapidly with increasing water binder ratios, the same trend is observed in consolidated fill (Monahan 1985).

The solids concentration is limited by the apparent viscosity of the slurry, which increases gradually with solids concentration up to some critical value (around 50% by volume for classified tailings fill), and then jumps dramatically with any small addition of solids. Fill is prepared so as to keep it a few percentage points below the critical concentration (fill preparation plants are not perfect and some allowance must be made for density fluctuations). If the apparent viscosity of the slurry is too high the head of pressure needed to move it may exceed the transport pipe line strength causing pipe bursting, or the available head may be insufficient to maintain flow velocity high enough to keep turbulent conditions so that the suspension settles and the line plugs.

Particle absolute size, density and distribution control the apparent viscosity of a slurry. Natural alluvial sands will generally require a lower solids concentration for transport than mill tailings fill. Increasing fines content will decrease apparent viscosity of a slurry at the same solids concentration; this is the principle behind the concepts of high density hydraulic and paste fills as discussed in Chapter 6. The subject of viscosity and slurry transport is a complex one.

For design purposes a solids concentration of 40 to 50% solids by volume is used. The coarser and more uniformly graded the fill material being considered the lower the solids concentration should be. An alluvial sand will commonly be placed at a lower solids concentration than a finer mill tailing hydraulic fill. Once in operation the fill solids concentration can be fine tuned to get the highest practical slurry density.

4.3.2 Flow Characteristics

Hydraulic fill slurries at rest are heterogeneous mixtures; they have a non uniform solids distribution with the larger or denser solids settling to the bottom of the mixture. However in transport, if the pipe velocity is high relative to pipe diameter, the flow will be fully turbulent and the slurry may be sufficiently mixed to achieve a pseudo homogeneous state, (solids uniformly distributed). In this state it can be described by the standard hydraulics equations for a Newtonian fluid. Practical calculations for the design of hydraulic fill distribution systems are based upon the assumption of turbulent flow and a pseudo homogeneous state.

Basic relationships governing the design of slurry systems, under the assumption of pseudo homogeneous state are:

1. Bernoullii equation (Conservation of Energy)

$$H + P/w + (V^2/2g) + H_f = C$$

where: H = vertical head at a point in the system, m
P = pressure, m of head
w = unit weight of fluid, kN/m^3
V = velocity of flow, m/s
H_f = head loss due to friction, m of head
C = a constant through out the system.

2. Darcy's equation

$$H_f = (f\ LV^2)/(D\ \ 2g)$$

where: f = friction factor, dependent upon velocity of flow, pipe diameter and slurry viscosity
L = length of pipe, m
V = velocity of flow, m/s
D = diameter of pipe, m
g = acceleration due gravity, m/s^2

3. Reynold's number, N_r

$$N_r = (\rho VD)/\ \mu$$

where: V = velocity of flow, m/s
D = diameter of pipe, m
μ = apparent viscosity of slurry, Pa.s
ρ = slurry density, kg/m^3

The Reynold's number is used to determine the friction factor from a Moody diagram. The relative roughness of the pipe, also required for determining the friction factor, can be found on standard tables. It should be noted that slurry flow will tend to smooth the inside of a pipe with time.

Fluid flow changes from laminar to turbulent flow at a Reynolds number of about 2100. Most hydraulic slurries will operate well into the turbulent flow regime. Calculation of the Reynolds number requires that the slurry apparent viscosity be known. Determination of slurry viscosity is not easy as it is dependent upon particle size distribution, solids density and various other factors. Spending time determining apparent viscosity is probably not worth while for conventional hydraulic fill system engineering as there is usually sufficient vertical head available not to have worry about friction losses. However for initial calculations, Thomas et. al. 1979 suggests a viscosity for hydraulic slurries about 30 time that of water, i.e. $\mu = 0.03$ Pa.s. The result is a friction factor generally in the range 0.040 to 0.015. A factor of 0.025 is usually suitable for initial pipeline calculations at typical slurry concentrations and velocities around 3 m/s.

For detail design work it is usual to determine a slurries friction factor with a test loop at the velocities, pulp density and temperature at which the system is expected to operate, so that calculation of Reynolds Number is not required. A plot of friction factor versus velocity based upon the test work is made from which calculation of pipeline pressures and flow rates etc is then straightforward. A better method is to take measurements from the actual system when first installed. Extensions to the system and replacement of worn lines can then be based upon more realistic engineering data.

4.3.2.1 Flow regimes

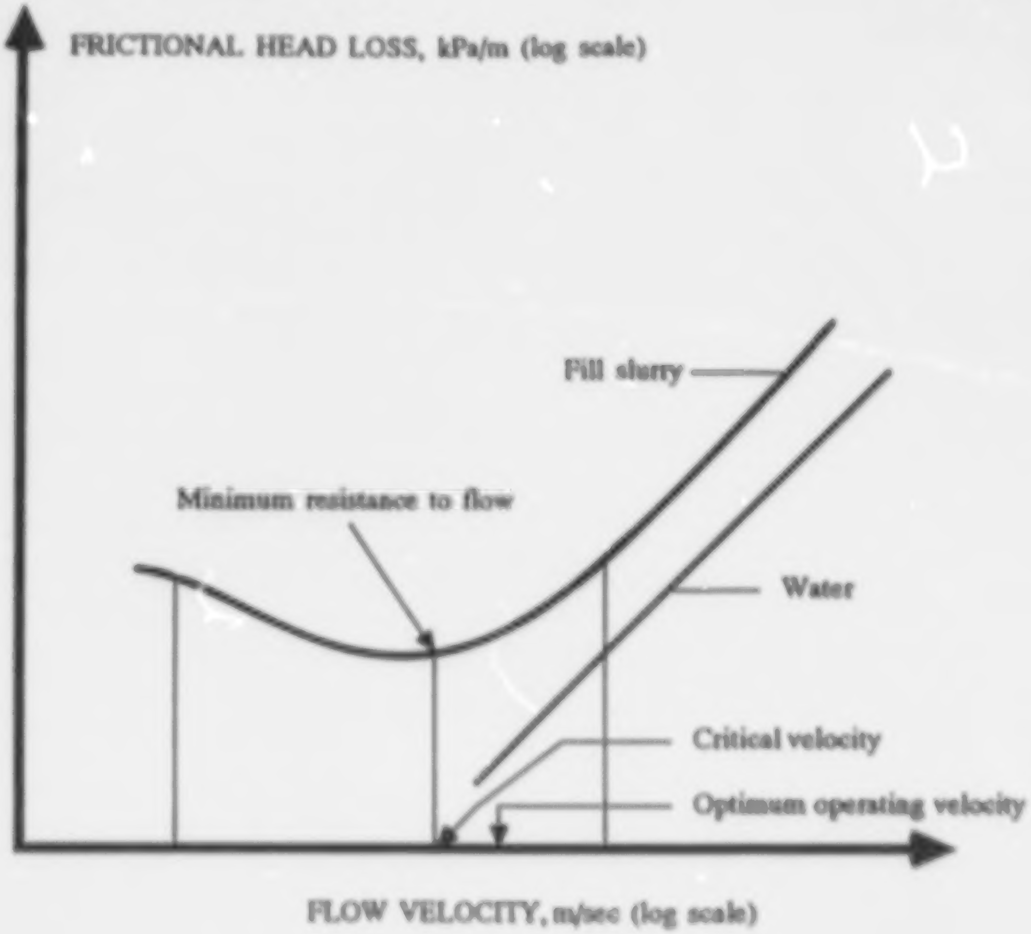
Although it is common to design fill slurry distribution systems based upon the assumption of pseudo homogeneous flow, in reality flow may not be so simple. Four flow regimes are recognized for backfill slurries. The flow rate or velocity determines which regime a particular slurry will flow in.

1. Stationary bed flow. Occurs at the lowest flow rates. The coarsest particles form a stationary bed of solids on the bottom of the pipe reducing the cross sectional area available for flow. Fine particles may remain in suspension. Line plugging at slope or direction changes is likely.

2. Saltation and sliding bed. The coarse particles are carried in a moving bed of solids at the base of the pipe or in a series of intermittent jumps called saltation. Line plugging at slope and direction changes and due to fluctuations in slurry density is common.

3. Heterogeneous flow. Solids concentration in the slurry is not uniform, tending to increase towards the base of the pipe.

4. Pseudo homogeneous flow in which the slurry is sufficiently agitated to act as a homogeneous liquid with uniform solids concentration. This occurs at high flow velocities.



Flow Regime	Saltation and stationary bed flow	Heterogeneous flow with sliding bed	Heterogeneous suspension. Operating regime for typical slurry transport,	Pseudo - homogeneous flow Assume this flow regime for design calcs.

Figure 4.4, Slurry flow regimes at increasing flow velocity.

The regime in which a fill slurry will flow at any point is dependent upon the solids specific gravity, liquid specific gravity and viscosity, particle size, shape, and distribution, flow velocity, pipe diameter and inclination, and the solids concentration. See figure 4.4.

4.3.2.2 Critical velocity

The critical velocity of a fill is the pipe flow velocity at which the fill particles will start to settle in the line. It is the velocity at which flow changes from heterogeneous flow to saltation and sliding bed flow. In practice this means the flow is slow enough to allow the larger (or heavier) particles in the slurry to start dropping out of suspension and form a sliding bed of material at the base of the pipe. A slurry may still flow under these conditions but fluctuations in flow velocity at changes of grade and bends may cause problems. At flow velocities below the critical, the fill line is prone to blockage. Fill distribution systems are designed to operate above the critical velocity.

As the proportion of fine material in a slurry increases the critical velocity tends to decrease. The fine particles tend to reduce the settling rate of the larger particles. This is the principle behind paste fills where the fines content is so high that flow may be stopped for some time without the larger particles settling out and blocking the line.

Critical velocities for slurries of deslimed mill tailings are typically 1 to 2 m/s, while for coarser natural sands typical values are 2 to 3 m/s. These typical velocities indicate that critical velocity is a function of particle settling rates. Coarser or denser particles will tend to settle faster under the action of gravity. At the high solids concentration of fill slurries the settling of particles is hindered by other particles in suspension.

The critical velocity is a function of pipe diameter. At a particular velocity, flow will be more turbulent in a small pipe than in a large pipe. The larger the pipe, the faster is the critical velocity.

The best known equation for calculation of critical velocity, V_c, is that of Durand.

$$V_c = F_L\sqrt{(2gD(s-1))}$$

where: F_L = constant related to particle size and volumetric solids concentration. Figure 4.5 gives approximate values for slurry concentrations 50 to 70% solids by weight.

g = 9.81 m/s^2

D = pipe diameter, m
s = specific gravity of solids

It should be remembered that Durand based his equation upon work done with dilute suspensions in comparison to fill slurries. For practical purposes critical velocity is best determined by test work or operating experience. The same pump loop tests used to measure pipeline friction factors may be used to determine the critical velocity of a fill slurry.

Critical velocity will tend to increase with the solids concentration of a conventional hydraulic fill. As the number of coarse particles increases the flow must become more turbulent to keep them in suspension. This effect is the reason for the common practice of reducing pulp densities where long horizontal transport distances are involved.

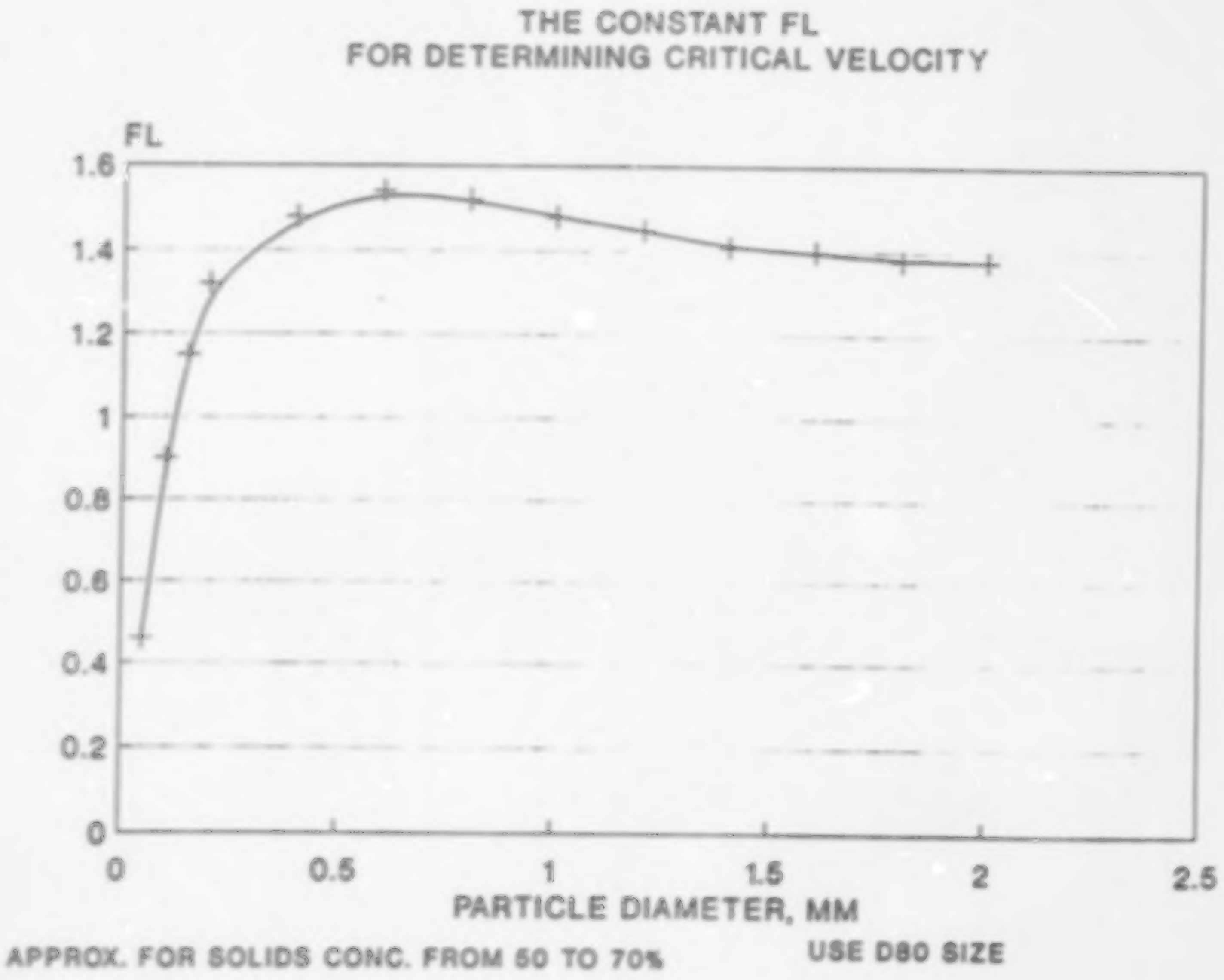


Figure 4.5 F_L versus particle size.
(after Koch, 1962 from Durand)

4.3.3 System Capacity

The rate at which a hydraulic fill can be placed in function of the efficiency of excess water drainage from the stope, the pressure which the distribution pipe can stand and acceptable pipe wear rates. In general we like to fill as rapidly as possible without exceeding these restrictions. Common placement rates are in the range 100 to 250 tonne per hour, with line velocities of 3 to 6 m/s.

Velocity of flow has the greatest influence on the system capacity. The pressure on a pipe increases with the square of the line velocity, and the wear rate is related to velocity by the power of 2 to 3. A high line velocity implies the need for an expensive high pressure pipe which will be subject to high wear rates. In order to minimize the cost of the distribution system, a hydraulic fill system is designed to operate at the minimum practical line velocity. This velocity is a compromise between capital cost of the pipe, system capacity and operating cost due to replacement and maintenance, a function of wear rates. In addition the velocity must exceed the critical velocity by some margin of error.

In order to maintain system capacity with a limited velocity of flow it necessary to keep the slurry density as high as possible and use the largest practical pipe diameters. Maintaining high slurry densities has other advantages discussed earlier. The pipe diameter used is restricted by what can be handled effectively in the underground environment, usually 150 mm diameter (6 inch). The cost of pipe must be weighed against the advantages of a higher system capacity. Frequently, because of drainage problems the extra capacity afforded by larger pipe can't be used or the additional capital cost can not be justified, so 127 mm (5 inch) or 100mm (4 inch) pipe is commonly used.

With pipes having a diameter in the order of 150 mm and flow velocities around 3 m/s it easy to place 300 tonnes per hour of fill underground with a reasonable slurry density, however the drainage facilities are usually not able to handle the quantity of excess water this rate implies. In placing hydraulic fill it is not good practice to let hydrostatic heads build up to dangerous levels, (100kPa or 5m head of liquified fill is the maximum permitted pressure on fill containment structures not classified as bulkheads, and is a good limit to hydrostatic heads in filling operations), nor is it good to place fill into ponded water because of severe segregation problems (especially consolidated fills). Hence the efficiency of drainage often is the controlling factor in the hydraulic system capacity. There are a number of ways in which drainage can be increased; filling two stopes simultaneously from the one fill line, increasing fill percolation rates, installing extra decant drainage facilities, others. These all take more manpower or extra cost, so it is usual to keep a simple system that limits the placement rate. Paste and high density hydraulic fills are being developed to overcome this drainage restriction.

In summary the procedure for calculating system capacity would follow the following path.

1. Select the highert possible solids concentration for the slurry. Say 70% for example, which gives a slurry density of 2.0 tonne/m^3 for a solid density of 3.1 tonne/m^3.

2. The slurry must flow at a velocity greater than the critical velocity, say 2 m/s for example.

3. From mine scheduling, mine pumping capacity and an estimate of stope drainage efficiency calculate the rate of fill placement required, say 200 tonne per hour for example.

4. Calculate volume of fill slurry required. 200 tph of 2.0 t/m^3 fill is 100 m^3/hour required.

5. Calculate maximum size pipe allowed to satisfy critical velocity restriction.
   100m^3/hour = 0.028 m^3/s
   At 2m/s the pipe cross sectional area required = 0.14 m^2
   This is a pipe inside diameter of 0.133 m
   Choose the next smallest practical pipe size, e.g. 102 mm ID

6. Calculate the actual flow velocity in the selected pipe, e.g 3.4 m/s

7. Calculate the pressure in the line expected at this velocity and choose the appropriate pipe material and strength rating. From this the cost of the system may be calculated.

8. Knowing the capital cost and estimating the wear rate due the calculated flow velocity decide if the original choice of system capacity is practical. Recalculate from step 3 to find the optimum capacity and pipe diameter.

4.3.4 Vertical Transport Problems

The extensive vertical sections common in hydraulic fill distribution networks can cause some interesting problems. Because the flow velocity in a hydraulic fill transport system must be regulated to avoid enorm line pressures and pipe wear rates, distribution systems are frequently operated only partially full (especially in deep mines).

Free fall conditions will occur in a vertical line when the resistance to flow in the system is lower than the vertical head of pipeline available. The slurry will only fill a vertical line far enough to overcome the resistance offered, flow from the preparation plant to this point will be by free fall. When a slurry free falls it will tend to entrain the air inside the pipe, creating negative pressures in the line. Water vapor evaporated from the slurry as it enters the line will compensate for these negative pressures. When the fill reaches the point of full flow at some depth in the line the pressure rises, forcing the water vapor to recondense violently. This may lead to cavitation erosion (high wear rates) and high transient pressures causing line vibration. Boreholes have been known to erode to several times their original size in a short space of time as a result of this affect. The erosion can be a major problem in pipelines installed in shafts.

Opening the top of the vertical borehole or pipe to atmosphere will prevent the development of low pressures in the line and hence the cavitation problem by allowing unrestricted volumes of air to enter. However, the large volumes of air will become entrained in the slurry and as the pressure in the line changes the entrained air will tend to occupy different volumes in the line. Hence the flow velocities will vary through the system as line pressure varies. In low pressure sections of the line the velocity will be greatest and hence wear rates will be highest. The line will be subject to surging flow and vibration, noticed particularly at the discharge point where pressure waves created by varying flow velocity may cause violent pipe vibration.

A better solution than allowing free entry of air, and one practiced at Falconbridge operations, is to allow only a controlled volume of air to enter the top of the vertical line so that the vacuum is only partially relieved, but is sufficient to prevent water vaporization and cavitation. A small hole (about 10mm) is drilled in the delivery pipe at the top of the free fall section which allows only a limited volume of air to enter the line. This limited amount of air will not cause large changes in the slurry volume and so flow velocities will be essentially constant through the system. If water hammer occurs the small amount of compressible air entrained in the slurry will help to dampen the over pressures. This solution will not completely stop the cavitation problem, but it will reduce it to a manageable level.

If the end of the free fall section occurs at the end of a borehole it is not uncommon for the casings to be frequently blown off. The system should be designed to keep the end of the free fall somewhere in the middle of the borehole, where the least damage can be done. Horizontal sections should be a minimum of 50 m long.

Water hammer occurs when the slurry speed is suddenly reduced. The kinetic energy of the slurry is reduced and the excess energy forms a stress wave which propagates upstream. When it reaches the end of the pipe or a direction change, a reflected tensile wave is generated which will vibrate the line. A number of devices can be added to the distribution system to eliminate the water hammer. One of the simplest is an air accumulator, simply a tank of air directly coupled to the slurry line. The compressible tank air helps absorb the energy of water hammer, dampening down the pressure waves. An accumulator tank has been used with success at Falconbridge, Sudbury Operations. Equations for the design of such accumulators may be found in fluid mechanics text books. Flow velocity should be kept as constant as possible throughout the system to minimize the potential for water hammer. The moderate compressibility of slurries containing entrained air helps to diminish the water hammer problem. Maintaining a full distribution line with no free fall sections has also been used to eliminate water hammer problems.

A common solution to all the vertical transport problems is to make the vertical lines run as full as possible. This is done by having small diameter vertical lines which give high flow resistance, installing horizontal pipe loops on intermediate levels in the vertical drop or installing restricting orifices to increase resistance to flow. These solutions are all designed to increase overall resistance to flow so that more of the available head is required to push the slurry and the vertical lines run full for more of their length. Falconbridge experience with horizontal pipe loops in a long vertical line suggest that pipe life can be increased 20 fold.

4.3.5 Bore Holes and Pipes

4.3.5.1 Bore holes

Most Canadian mines rely upon bore holes to pass hydraulic fill from surface to some central underground location where it is distributed by horizontal pipe and further bore holes to the placement location. Bore holes will be unlined where ground conditions allow. Underground fill holes are sealed at their collars using steel breastplates. The breastplates are rock bolted in place and grouted with quick setting cement. Where ground conditions are poor, fill boreholes can be lined with wear resistant ABS pipe or steel liners to prevent inside damage. Steel liners are grouted in place and can be further lined with an inner wear-resistant fill pipe if backfill material is very abrasive.

The borehole diameter is often less than the horizontal pipe diameter. A smaller vertical hole results in less air entrainment during free fall conditions (less area for the air to occupy) with all the advantages this implies. A smaller hole will tend to run fuller than a large hole (smaller hole implies higher velocity and hence greater friction losses) which will assist in keeping flow steady, reducing water hammer and surging in the line, and reducing cavitation erosion.Smaller holes are cheaper to drill as well. 76 mm (3 inch) bore holes are not uncommon.

When conditions are such that the vertical fill delivery line must be a pipeline rather than borehole then every effort should be made to keep the line as full as possible. Full flow will give better wear characteristics. Methods of achieving full flow in addition to using smaller diameter pipes have included additional horizontal loops on intermediate levels to increase pressure loss, and introduction of orifices into the line to increase friction head loss. Preventing cavitation erosion is particularly important in vertical pipelines which may often be in critical areas of the mine such as the shaft.

4.3.5.2 Type of pipe

The most common pipe used for fill transport is unlined steel pipe. In places where pressures are low, high density polyurethane (HDPE) pipes have become popular for their low cost and wear resistance. Rubber lined steel pipe is commonly used when appreciable tonnages are involved. It provides good wear resistance with the strength of steel for high pressure applications (Ashby & Hunter 1982). Rubber has a tendency to peel and form "flappers" which can later clog lines, especially when subject to cavitation erosion. Rubber is also more susceptible to damage from gravel than polyurethane (Hull, 1978). Polyurethane lined pipes have been used in place of rubber lined pipe and show good wear characteristics.

4.3.5.3 Pipe wear

Wear in pipes is a function of both corrosion and abrasion. For fill slurry lines, abrasion is the primary cause of wear. Abrasion rates are controlled principally by flow velocity, but also by solids concentration, and particle absolute size and range. Practical experience suggests that pipe wear is directly proportional to solids concentration and proportional to velocity raised to the power of 2 or 3. Fill distribution systems are designed to operate at the lowest possible flow rate; faster than the critical velocity, fast enough to deliver the required tonnage of fill, but no faster, so that wear and line pressures are minimized.

Wear is most severe at direction changes. It is common to use wide sweep bends or tee pieces at direction changes. Falconbridge has found tee pieces to be effective at minimizing wear at bends subject to moderate tonnages. The blank arm of the tee piece collects a bed of fill which protects the metal from abrasion. Ceramic lined fittings may also offer a solution to excessive wear at bends and tees in more permanent installations with high tonnage requirements. High pressure flexible rubber hoses are used for direction changes at the bottom of boreholes and for easy line changes.

Wear in fill delivery systems is also concentrated in regions where there are long vertical drops. The critical distance for vertical runs at which wear markedly increases is 50 meters (Ashby & Hunter 1982).Another area where high wear is often seen is in short sections of horizontal fill line between two vertical drops. Minimum horizontal runs between verticals should be 50m.

It is common to rotate long horizontal sections of fill line to ensure equal wear around the full circumference of the pipe. Fill is more abrasive on the bottom of a pipe. Rotating the pipe will spread the wear and maximize pipe life.

4.4 DRAINAGE

Conventional hydraulic backfill contains 25% to 50% water by weight. If the fill is to be consolidated, only a small fraction of this is required for hydration of the cement. Excess water poses a threat to safety due to the resulting high head pressures which may be exerted against fill barricades. Failure of the barricade can result in uncontrolled fluidized flow of backfill into the mine's access areas, a hazardous condition for workers and equipment and a severe mine drainage problem. Hence, prior to hydraulic filling, provision must be made for the drainage of excess water and safe containment of fill in the stope. At the same time, unfiltered mass drainage of excess water from the stope can cause excessive loss of fines and binder with the drain water. Therefore, it is essential to provide drainage that is filtered and controlled.

4.4.1 Dewatering Mechanisms

Three fill dewatering mechanisms are recognized (Thomas et. al. 1979). These are;

1. Decantation from the upper surface of the fill.

2. Exudation of water from the body of the fill due to consolidation followed by decantation.

3. Percolation through the fill under the action of gravity.

Figure 4.6 illustrates these mechanisms.

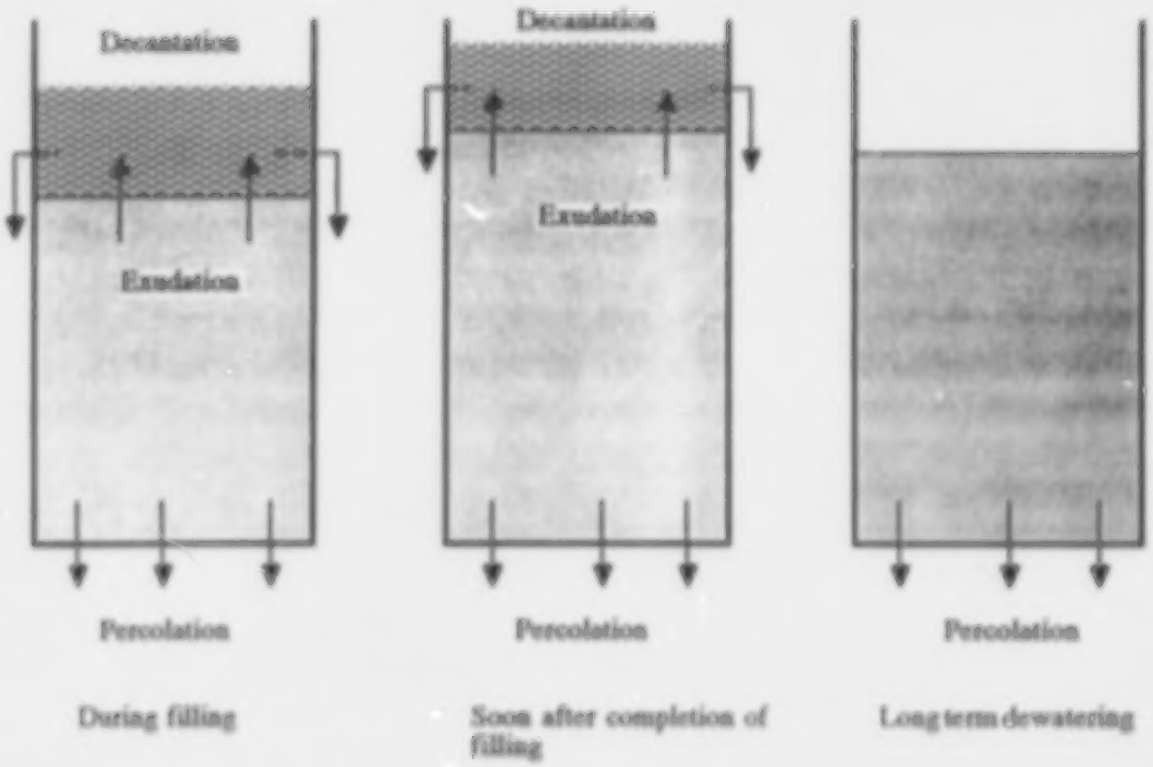

Decantation
Exudation
Percolation
During filling
Decantation
Exudation
Percolation
Soon after completion of filling
Percolation
Long term dewatering

Figure 4.6, Hydraulic fill dewatering mechanisms. (after Thomas et. al., 1979)

4.4.2 Practical Drainage Methods

In practice drainage of fill water is a combination of the three mechanisms described above. The need to dewater fill as rapidly as possible encourages the use of every available drainage opportunity. Decantation is perhaps the most effective mechanism; over 80% of the fill water which is ultimately removed from a stope can be by decantation. Longer term dewatering relies upon percolation drainage, and some operations rely solely upon percolation drainage for dewatering. Installing provision for percolation dewatering is expensive, time consuming and sometimes not possible. The drainage pipes, decant towers, and drain raises employed are designed to facilitate both decantation and percolation drainage.

4.4.2.1 Drainage pipes

Drainage or decant pipes commonly used in blast hole stopes are 100mm or 150mm diameter perforated plastic or metal pipes (common agricultural drainage pipe). The pipe is covered with filter cloth to prevent fines escaping from the fill with the water. The pipes are secured by pinning at the top of the stope to be filled. They extend down through the entire stope height and are braced at the bottom by being placed directly through the fill barricade, thereby forming part of the fence construction. The size and shape of the stope determines how many drains will be used. At least two are used in case one should become blocked. Figure 4.7.

Drainage pipes extending to the fill surface facilitate both decantation and percolation drainage. Decantation is the their initial role, collecting the free water at the fill surface. In the medium term after free water is removed they provide a free face attracting percolation draiinage water.

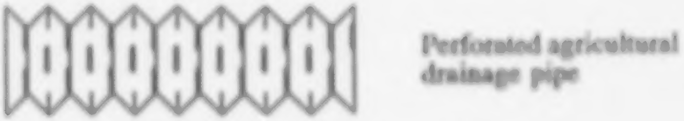
Perforated drainage pipes are a preferred dewatering system for the following reasons (Soderberg & Busch 1985):

1. Water will not accumulate to a hazardous depth in the stope at any time when the pipes are functioning properly.

2. Water removal starts as soon as filling starts.

3. Water clarification is not necessary before removal.

4. No workers are required to work the system.


Perforated agricultural
drainage pipe

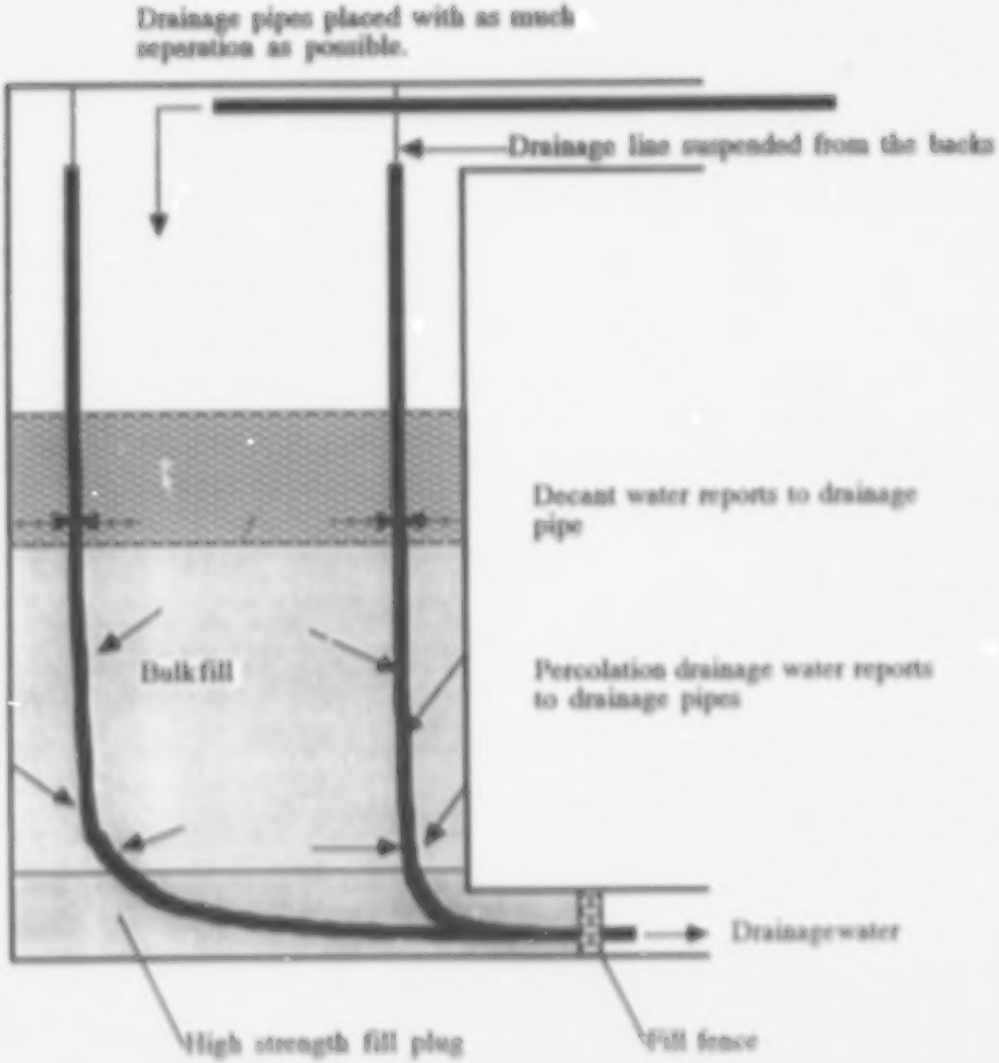

Drainage pipes placed with as much separation as possible.
Drainage line suspended from the backs
Decant water reports to drainage pipe
Bulk fill
Percolation drainage water reports to drainage pipes
Drainagewater
High strength fill plug
Fill fence

Figure 4.7, Drainage pipe installation in a blast hole stope.

5. Filling can be continuous if desired (i.e. it is not necessary to suspend filling to wait for gravity drainage to occur).

Drainage pipes should be located at the corners of a stope away from the pour point where excess water will collect. The less excess ponded water in the stope the better will be the fill quality (reduced segregation). So when access allows, drainage should be located anywhere water is likely to pond.

4.4.2.2 Decant towers and raises

Decant towers are often used to enhance drainage in smaller cut and fill stopes. Towers are usually erected over a stable base; bricks and mortar or strong timber. The body of the tower is essentially just a large permeable column of filter fabric lightly supported. It can be constructed in a variety of ways. Light construction is sufficient if good permeability is maintained. A simple tower can consist of filter cloth wrapped around tightly rolled 150 mm square wire mesh. Timber can also be used to provide additional support as needed. The decant tower is either connected to a drain raise by a pipe, or it can contain drain pipes which will collect and carry excess water through the filter plug and fill barricade at the base openings of the stope. Although towers can extend to varying heights within the stope, they are usually positioned where they can provide drainage of the "earliest reporting water" (Ashby & Hunter 1982). Figure 4.8.

Raises carried through the fill in the stope or in adjacent pillars can be used for effective fill drainage. A rise carried through the fill in a stope will be wrapped in filter fabric and possibly wire mesh for additional support to the fabric. The rise will then act as a large drain pipe. When raises are present in adjacent ground a series of small diameter holes can be drilled across into the fill to facilitate dewatering.

4.4.2.3 Pumping

Mechanical pumping has been used to dewater fill. Here, the fines in the backfill material are given enough time to settle and the layer of clear water formed at the top of the fill is pumped off. This method is not conducive to good backfill placement or efficient dewatering. It is difficult and dangerous when open stopes are being filled. The practice is inefficient because of the dirt and grit which enters the pump, requiring frequent maintenance (Soderberg & Busch 1985). The failure of normal drainage methods often forces operators to revert to pumping; for example when all drainage pipes are broken or blocked. Stopes at the bottom of a mine with no development underneath may also require pumping.

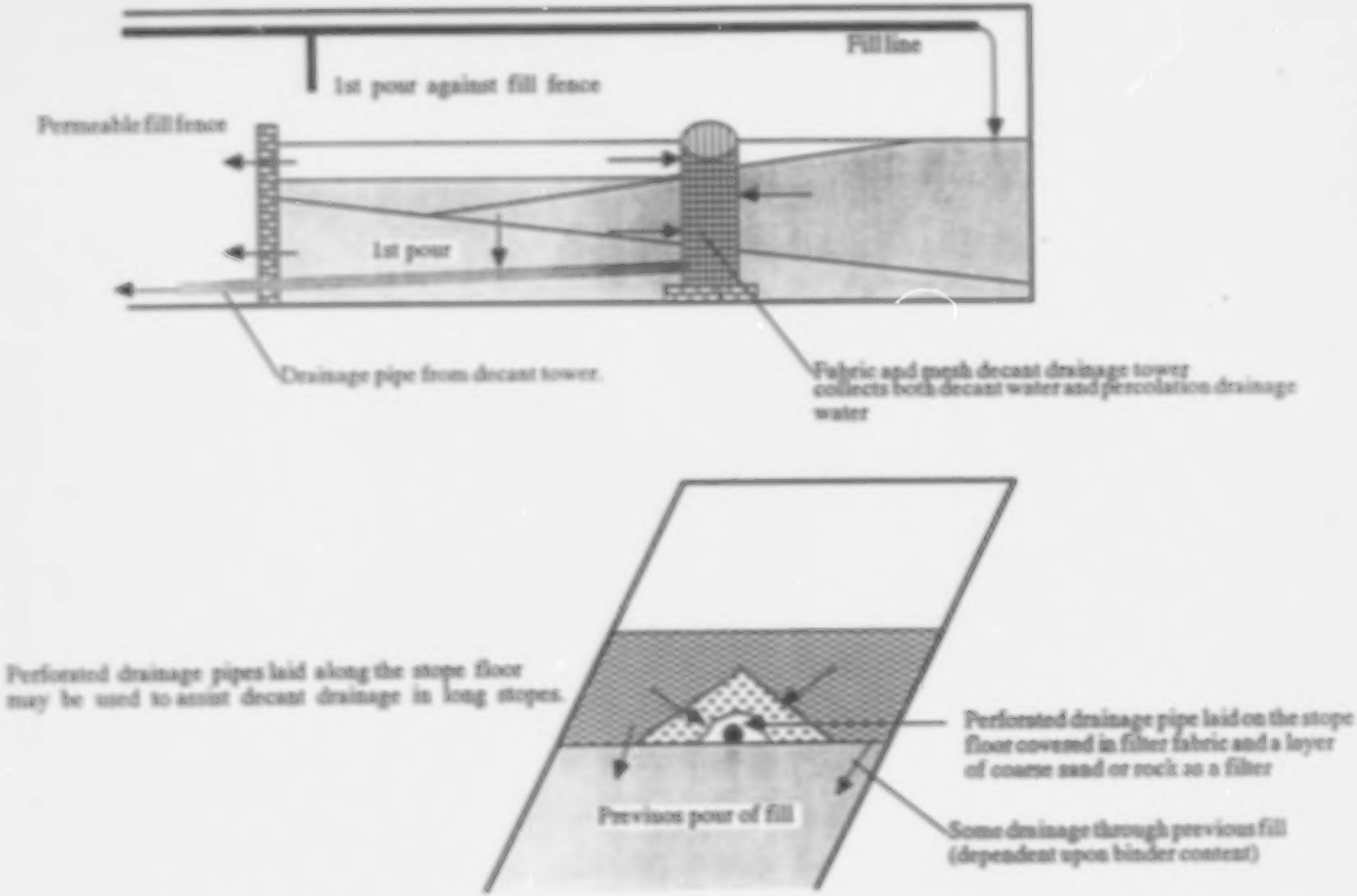

Fill line
1st pour against fill fence
Permeable fill fence
1st pour
Drainage pipe from decant tower.
Fabric and mesh decant drainage tower collects both decant water and percolation drainage water
Perforated drainage pipes laid along the stope floor may be used to assist decant drainage in long stopes.
Perforated drainage pipe laid on the stope floor covered in filter fabric and a layer of coarse sand or rock as a filter
Previous pour of fill
Some drainage through previous fill (dependent upon binder content)

Figure 4.8. Application of decant tower and perforated drainage in stope dewatering.

4.4.2.4 Electrokinetic dewatering

An innovative technique for dewatering hydraulically filled stopes is electrokinetic dewatering. This method was proposed by the U.S. Bureau of Mines in the mid-1970's. Although there has been significant laboratory and in-situ testing in deep underground mines, there is no indication that the method has ever been adopted as standard practice by any operating mine. In this technique, electrodes are buried in backfill. If perforated metal drain pipes are normally used for dewatering hydraulically filled stopes, these can double as electrodes. Alternatively, wire mesh can also be used. When electrical power is passed through the electrodes in the form of direct current, slow settling fine particles (i.e. slimes) can be dewatered and densified to form competent backfill. The method is claimed to be safe and efficient for use in underground hydraulically filled stopes. It is described in detail in Sprute and Kelsh (1975), Sprute and Kelsh (1976), and Sprute and Kelsh (1978).

4.4.2.5 Consolidated fill drainage

Consolidated hydraulic fill pours present some difficult dewatering problems because of the rapid decline in mass permeability as the cementing reactions proceed. Decantation tends to play a more critical role in consolidated fill dewatering. Permeability of cured low cement content fill (e.g. 32:1) is about 1/4 of the unconsolidated fill values. As the amount of binder added increases the permeability drops rapidly. A 10:1 consolidated fill will be essentially impermeable and water may pond on its surface. This lack of permeability is not a long term problem as the internal cohesion of the consolidated fill will preclude the chance of liquefaction.

The most critical period for consolidated fill permeability is during the first 12 hours after placement when drainage capacity is most important. The permeability of uncured fill (during the first 12 hours) is related to the amount of fine binder particles added to the fill mix. The higher the binder content the lower the percolation rate, just as if the fill had not been classified to remove the slimes fraction. At high binder addition rates the early percolation rate of a fill can be expected to be significantly reduced (approximately 50% of the fill without binder added). Decantation must be expected to play a greater role in the dewatering of consolidated fills. It is important that as much excess water is removed from a consolidated fill as possible so that the best water binder ratio is realized and hence optimum strength developed.

The true nature of drainage through cured consolidated fill is a matter for some investigation. It is possible to get worm hole effects in consolidated fill, where open drainage channels established during initial stages of pouring are in set in place in the fill as it cures. These may lead from close to the fill surface to the drainage barricade and assist in long term drainage, acting much like installed drainage pipes. They may also be responsible for fill barricade failure because they can provide a direct path for hydraulic pressure to act on the bulkhead.

4.4.2.6 Pour rates

It is common to suspend filling operations several times in order to allow transport water the time required to drain sufficiently so that excess hydrostatic pressures do not build up. Once a sufficient quantity of water has drained from the stope, pressures become minimal and filling can be resumed. Obviously the better the installed drainage system the faster the fill can be poured and the less interruptions required.

4.4.3 Filter Fabric

Filter fabrics are used to minimize the loss of fines and binder particles from the stope with the fill drain water. Fines cause many problems in a mines dewatering system and loss of binder represents a considerable expense and reduction in the fill strength. Commonly the drainage pipes and decant towers will be wrapped in a suitable fabric which will allow the passage of water but trap the fine particles. A variety of natural and synthetic materials are suitable for use as filter fabrics. The selection of a particular fabric is a compromise between the speed of drainage and the degree of filtration. Ordinary Hessian cloth was one of the first filter fabrics used.

4.5 FILL CONTAINMENT BARRICADES

Prior to hydraulic filling, it is necessary to construct barricades across openings into the stope in order to retain the fill. The required strength of such retaining structures varies with each application and depends on the pressure which is expected to build up behind it.

There are two approaches to containment barricade design.

1. Impervious containment which is designed to withstand the full potential head of liquefied fill. An impervious structure must be very strong. It may have additional drainage installed to facilitate flow of water through the structure in order to minimize liquefaction potential, but these are not critical to the design.

2. Free draining containment which is designed to ensure a well drained fill and hence no liquefaction . These structures need only resist earth pressures and moderate hydrostatic pressures required to ensure water flow and can be of moderate strength.

The single most important fill property affecting the barricade design is whether the fill is consolidated or not. Any structure containing consolidated fill needs only withstand the load until the fill has set and developed some true cohesion, which may be only a matter of hours. Unconsolidated fill represents a much greater hazard potential and the containing structure must be designed considering the potential hydrostatic head from liquefied fill.

4.5.1 Legal Definitions

A fill containment barricade can be a simple fill fence of light construction or a strongly built bulkhead. The term "bulkhead" has sometimes used synonymously with "fill fence". Discretionary use of the term "bulkhead" has been encouraged in Ontario because of labour regulations regarding the installation of such structures. There are certain legal requirements associated with the use of bulkheads which do not apply for fill fences. Whether or not the stipulations apply depends on the technical definition of the term "bulkhead" and whether or not a given fill barricade qualifies as a bulkhead.

The term "bulkhead" has previously been defined by the Ministry of Labour of Ontario as "a structure for the impoundment of water or compressed air in an underground opening and constructed so as to completely close off the opening" (Ministry of Labor of Ontario 1978). Since the advent of bulk mining and very large fill pours, the Ministry has revised its definition to "a structure for the impoundment of water, compressed air, hydraulic backfill or any material in an underground opening where the potential pressure against the structure will be in excess of 100 kilopascals" (Ministry of Labor of Ontario 1987). This change has been made to avoid confusion regarding the submission of technical drawings, plans and specifications for the construction of bulkheads and the posting of notices as provided for in the Occupational Health and Safety Act of Ontario (Ames 1989). Under the revised ministry definition, backfill bulkheads are considered to be only those structures which retain an amount of undrained backfill in excess of approximately 4.5 m (15 feet) vertically, equivalent to a pressure of 100 kilopascals (Ames 1989). Once the fill is cured or drained it will not exert hydraulic pressure. As a result, it is not necessary to supply drawings and specifications for the smaller scale barricades which are used on a standard basis in combination with a fill plug (described below).

4.5.2 Impervious Fill Containment

One design approach to fill containment requires the construction of a structure capable of withstanding the maximum load due a stope full of liquefied fill. The pressure due liquefied fill may be calculated as;

$$P = wh$$

where: P = hydrostatic pressure due to liquefied fill, kPa
w = unit weight of saturated fill, 24 kN/m^3
h = height of liquefied fill, m

4.5.2.1 Containment Plugs

A containment plug is a plug of consolidated fill poured at the base of a stope which has sufficient cohesion to retain potential liquefied fill in the stope above. Figure 4.9 illustrates the concept. There are two variations on the concept, the drift plug and the full bottom plug. For ease of placement the full plug is more often used.

A plug will require some containment of its own while it is being poured. Light construction fill fences are used for this job. A great variety of fill fence designs are in use, each operation having their own favorite. Chapter three presents a small selection of fence designs. Plugs are poured in three or four stages to limit the hydraulic head on the fence. Each layer is given time to cure before the next is poured.

The design of a simple drift plug is the easiest to analyze. Here the cohesion between plug and drift must be sufficient to resist the force due liquefied fill acting on the plug. The plug to drift cohesion is assumed to be equal to the internal cohesion of the consolidated fill.

$$F = whA_{plug}$$

where: F = force acting on plug due to liquefied fill, kPa
w = unit weight of saturated fill, 24kN/m^3
h = height of liquefied fill in the stope, m
A_{plug} = cross section area of drift plug

The resisting force **R**, is that due cohesion between fill and the floor and walls of the drift (we don't consider backs of the drift because fill settlement after placement may result in poor contact).

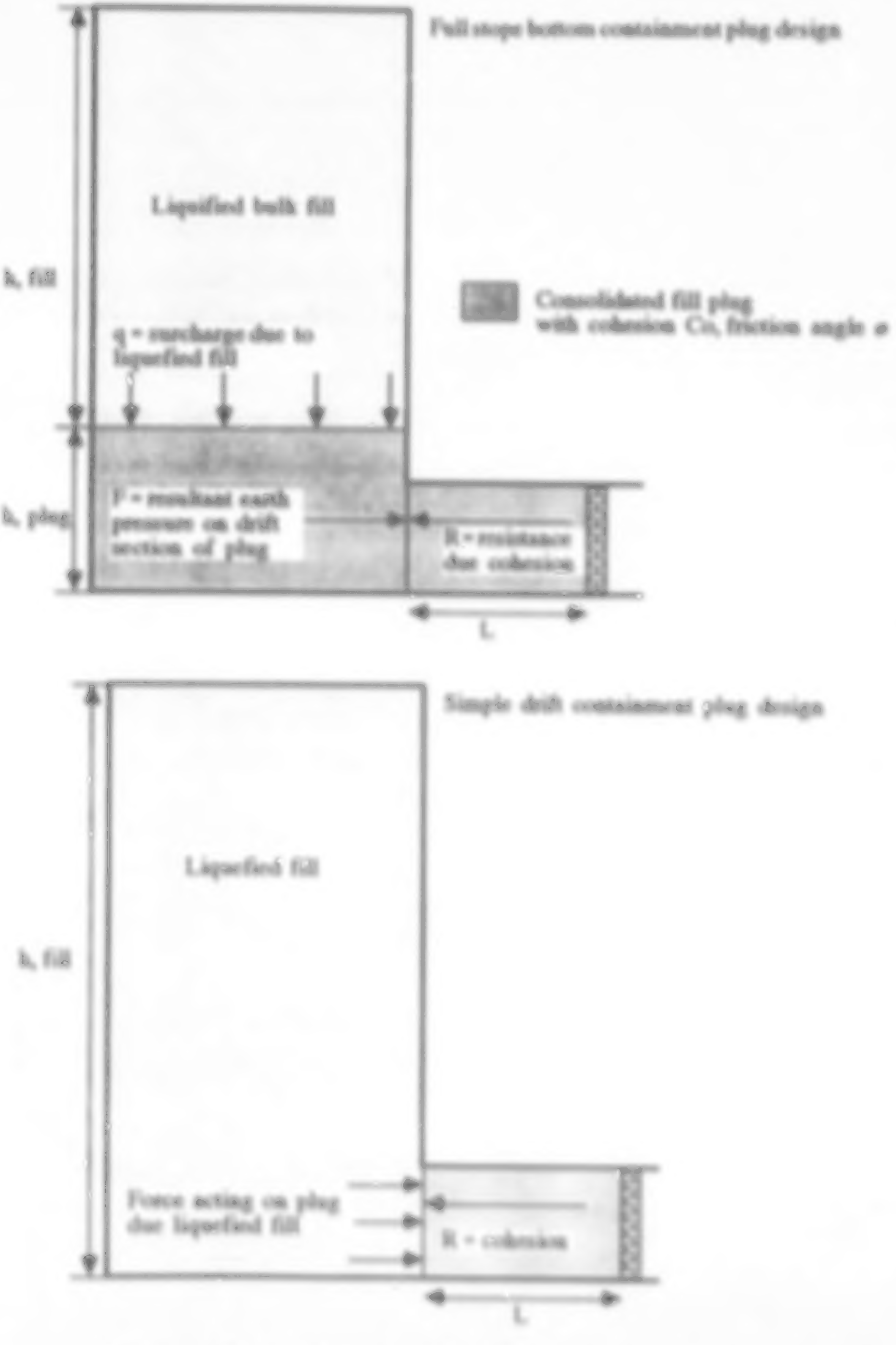

Full stope bottom containment plug design
Liquified bulk fill
h, fill
Consolidated fill plug
with cohesion Co, friction angle ø
q = surcharge due to
liquefied fill
F = resultant earth
pressure on drift
section of plug
h, plug
R = resistance
due cohesion
L
Simple drift containment plug design
Liquefied fill
h, fill
Force acting on plug
due liquefied fill
R = cohesion
L

Figure 4.9. Containment plug designs.

$$R = 2C_o h_{plug} L + C_o W L$$

where:
C_o = plug fill internal cohesion, kPa
h_{plug} = height of plug, m
L = length of plug, m
W = width of plug, m

So long as R exceeds F then the plug should be stable. It is important to remember that the internal cohesion of the plug fill will require some curing time to develop, and pouring the bulk fill should be scheduled to permit sufficient cohesion to develop.

A full stope bottom plug is a more stable arrangement because the force of liquefied fill must act through the consolidated fill in the stope to apply pressure to the drift plug section. Rankine's theory of earth pressure can be used to calculate the force F, which will be acting to move the drift plug section.

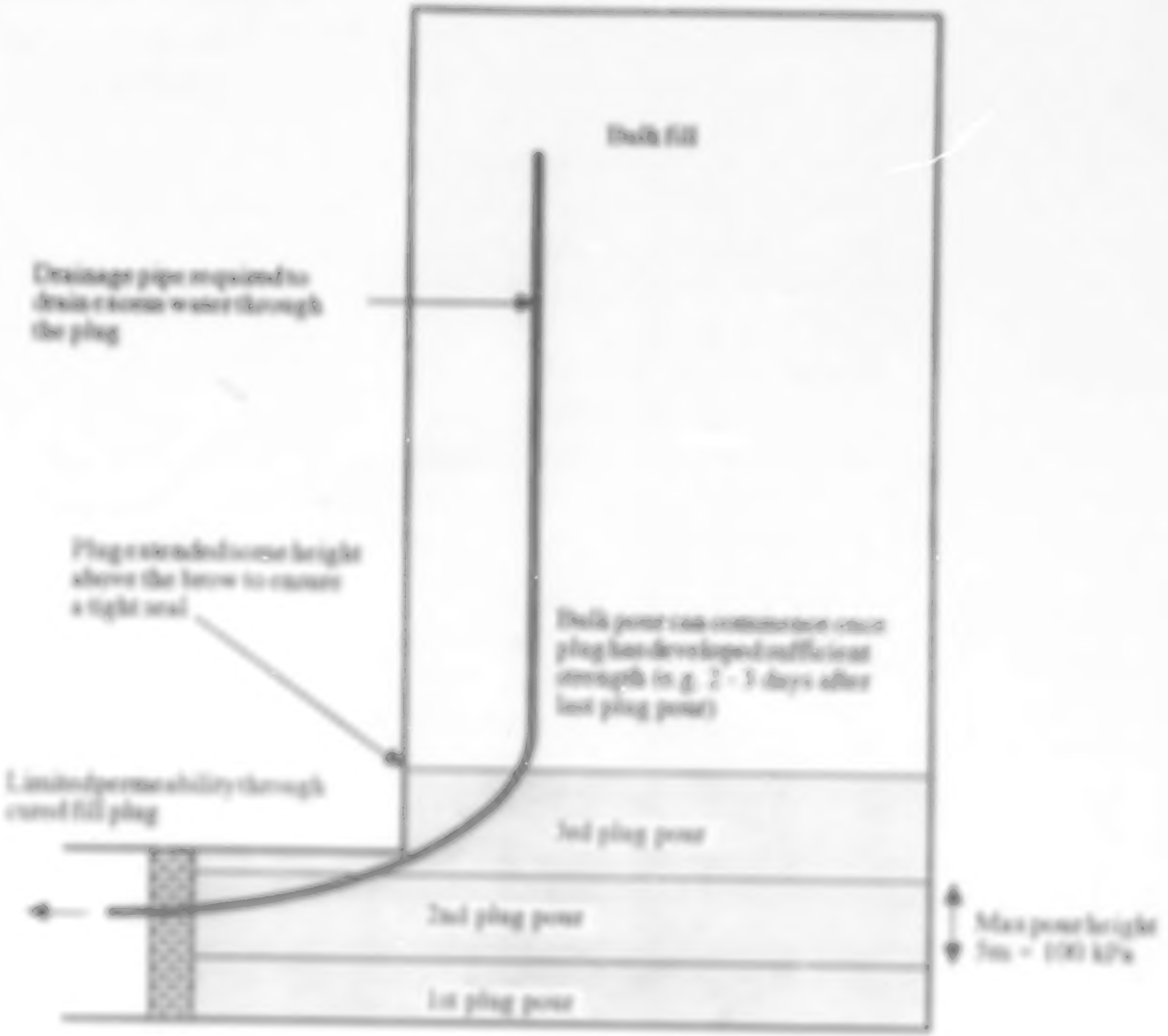
$$F = A_{plug} (w_{plug} h_{plug} K_A + qK_A - 2C_o\sqrt{K_A})$$

where:
K_A = $(1 - \sin\phi)/(1 + \sin\phi)$
ϕ = friction angle of fill
q = $w_{lf} h_{lf}$ the surcharge load on plug due liquefied fill
w_{lf} = unit weight of saturated fill, 24 kN/m^3
h_{lf} = height of liquefied fill, m
A_{plug} = cross section area of plug on which disturbing force acts
w_{plug} = unit weight of consolidated fill in the plug, kN/m^3
h_{plug} = height of plug, m
C_o = internal cohesion of plug fill, kPa

The disturbing force on the plug in this design is much lower than for the simple drift plug design, while the resisting force is calculated in the same way. The full stope bottom plug is easier to pour because it requires only one fill fence.

A containment plug relies upon the provision of auxiliary drainage facilities for dewatering of the bulk fill during pouring. A plug at Falconbridge's Sudbury operations is usually poured at a tailings cement ratio of between 10:1 and 16:1. Lower ratios have been used when a adequate length of drift is available.

fig 4.10


Bulk fill
Drainage pipe required to drain excess water through the plug
Plug extended some height above the brow to ensure a tight seal
Bulk pour can commence once plug has developed sufficient strength (e.g. 2 - 3 days after last plug pour)
Limited permeability through cured fill plug
3rd plug pour
2nd plug pour
1st plug pour
Max pour height 5m = 100 kPa

High strength fill containment plug poured in three lifts to prevent excess hydrostatic pressure build up against the fill fence during the pour, each pour is given 24 hr to cure before the next is poured.

Figure 4.10, Detail of containment plug in a blasthole stope.

4.5.2.2 Massive Bulkheads

Massive bulkheads were the original design approach to fill containment in large stopes. These are designed to withstand the full potential liquefied fill pressure with no drift plug. Auxiliary drainage is often installed to limit the load on the structure, but these are not critical to the design (i.e. structure designed to stand independent of the functioning of any drainage installed). In practice these structures will be holding only a fraction of the load they have been designed for, unless some fill liquefaction occurs of coarse.

Design of a massive bulkhead is a simple structural calculation based upon the shear and bending strength of the construction material. A typical massive bulkhead would be constructed of reinforced concrete or concrete block bricks hitched to the drift walls. Timber bulkheads are possible for smaller stopes. Figure 4.11 is an idealized design. Construction of such massive bulkheads is expensive and time consuming.

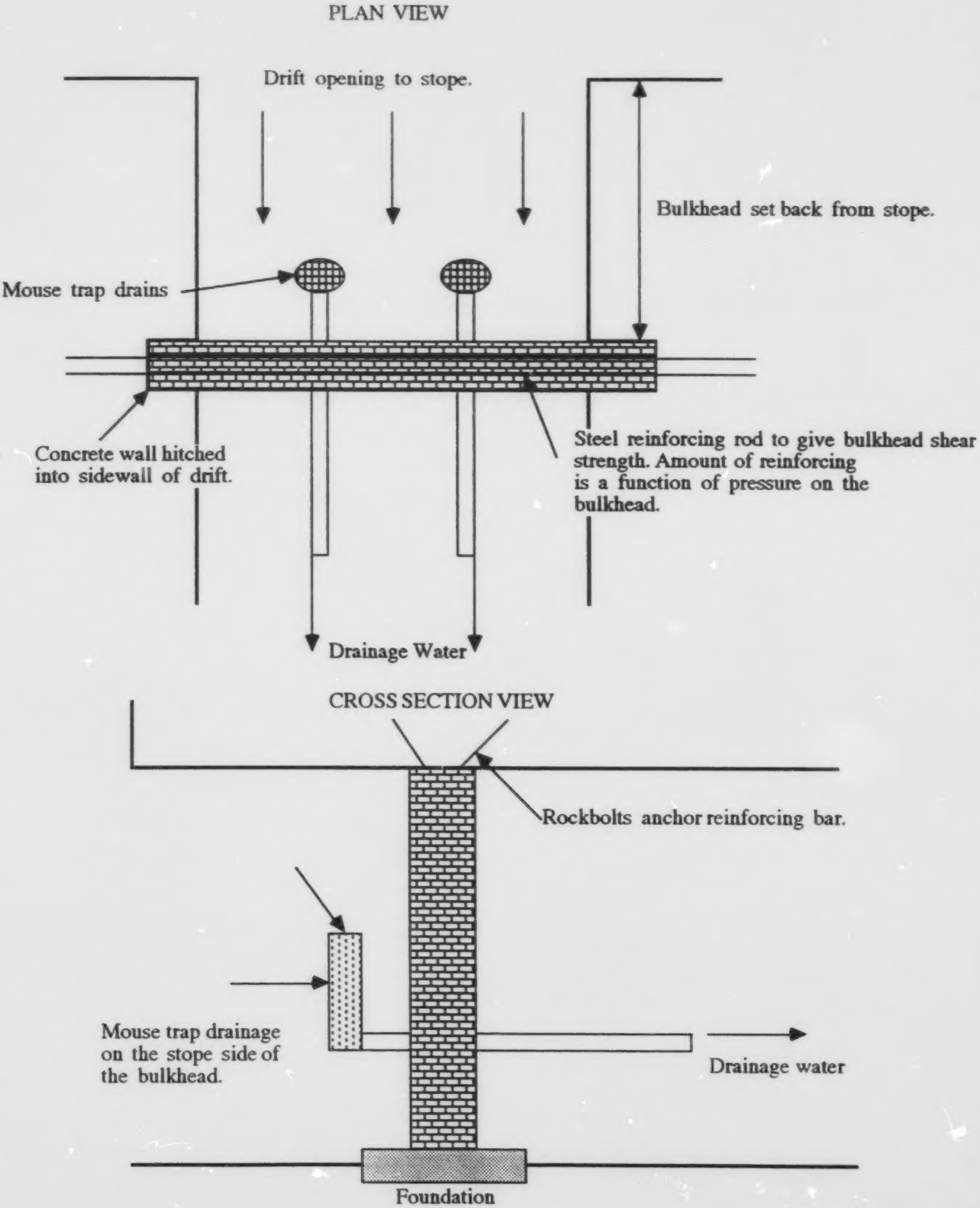

PLAN VIEW
Drift opening to stope.
Bulkhead set back from stope.
Mouse trap drains
Concrete wall hitched into sidewall of drift.
Steel reinforcing rod to give bulkhead shear strength. Amount of reinforcing is a function of pressure on the bulkhead.
Drainage Water
CROSS SECTION VIEW
Rockbolts anchor reinforcing bar.
Mouse trap drainage on the stope side of the bulkhead.
Drainage water
Foundation

Figure 4.11, Typical concrete bulkhead design

4.5.3 Free Draining Containment

A free draining containment structure design is based upon the assumption that by maintaining a well drained fill there will be no potential for liquefaction. A containment barricade may then be constructed which must only withstand low to moderate pressure.

The total pressure acting on a barricade can be separated into two components (under the assumption that liquefaction is not possible).

1. Solid fill pressure expressed as active earth pressure.
2. Hydrostatic water pressure.

Solid fill pressure is a result of fill weight less arching support and cohesion support in consolidated fill. A consolidated fill will be essentially self supporting and not exert any solid pressure. For unconsolidated fill the effect of arching in both the drift and the stope is important in reducing the solid fill pressure. Limited work by Askew et. al. (1978) indicated pressures in the region of 20 kPa acting on fill barricades independent of stope height. These authors recommend a design pressure of 74 kPa for free draining bulkheads set back 75% of the drift width (discussed in Smith and Mitchell, 1975). This assumes the drift plug section of the fill remains unsaturated and acts like a soil which is capable of developing arching. Should the soil become saturated it may no longer be able to support arching.

The hydrostatic pressure at a free draining containment structure will be zero, but there will be some pressure on the stope side of the drift plug. This pressure will equal the head required to push water through the drift plug and containment structure. This pressure will act to dislodge the drift plug and should be considered as an additional load on the barricade (Smith and Mitchell, 1975). Smith and Mitchell propose the following equation for determining the water pressure on a free draining barricade.

$$u = (QSw)/(PA)$$

where:
u = water pressure on barricade, kPa
P = percolation rate of drift plug material, m/hr
Q = drainage quantity , m^3/hr
A = total area of drainage drifts, m^2
S = length of drift plug, m
w = unit weight of water, 9.18 kN/m^3

The total design pressure for a barricade would be 74 + **u** kPa. The value of **u** will generally be much less than the solid fill pressure component especially once most of the fill water is drained and Q is very small.

4.5.3.1 Free Draining Bulkheads

It is relatively simple to design bulkheads to withstand the moderate pressures involved in the free draining containment approach. Timber bulkheads are typically designed to resist pressures in the region of 210kPa. Simple concrete brick or arched brick designs are also possible. These structures will have gaps left to allow water flow and be covered in filter [illegible] The main problem is ensuring the continued good drainage by placing highly permeable rock or coarse sand in the drift plug. Auxiliary drainage pipes or towers may also be used.

4.5.3.2 Drainage Blankets

Drainage blankets are unconsolidated containment plugs which rely on material arching rather than cohesion to resist hydrostatic head. Sometimes, drawpoint cones in open stopes to be hydraulically backfilled have been filled with broken rock which acts as a permeable plug. The rest of the stope is then filled with regular hydraulic fill which is contained by the broken rock. This practice is suitable for some operations, but the large voids present between rocks can lead to excessive loss of fill during initial pouring. A drainage blanket is a refinement of this practice that provides better filtration and less fill loss.

A detailed paper on fill drains and barriers (Smith & Mitchell 1982) identifies several types of non-cemented permeable drainage blankets. Blankets consist of a combination of fragmented, non fine-producing and non-decomposing waste rock and well graded medium to coarse sand. The drainage blanket still requires a fill fence for confinement of the initial pours. Several possible combinations of drainage materials placed for the blocking of a large stope opening are shown in Figure 4.12. The combination selected will depend on site-specific factors such as material availability, practicality and economics.

The drainage blanket is not a popular fill containment method. It requires good control in placement of the plug material to meet the design drainage requirements. It also requires that the bulk pour be well managed and monitored to ensure adequate drainage is being maintained. The degree of control required is often difficult to maintain in the production environment. Even though the design suggests that there is an adequate factor of safety when using drainage blankets, problems with practical implementation may quickly reduce the factor of safety without management quality control. The incentive for the use of drainage blankets is low cost because high cost binders are not required and it does not require a backfill system capable of placing consolidated fill.

4.5.3.3 USBM Plug

An idealized design for a drain plug and fill fence combination for large VRM or blasthole stopes has been proposed by the U.S. Bureau of Mines (Soderberg & Busch 1985). The plug system is shown in figure 4.13. The main features of the design are:

1. The design applies to cemented or uncemented tailings or sand filled stopes.
2. To prevent clogging of the filter, the drawpoint must be thoroughly cleaned by washing to where the cone starts.
3. The fill fence should be placed at a minimum distance of 1.5 times the crosscut height outside the brow of the drawpoint, and it should be anchored by hitches cut into both ribs and the floor of the crosscut.
4. The drain plug consists of the installation of perforated drainage pipes in a 0.9m thick layer of clear stone or gravel, 16 mm to 51 mm size (5/8 to 2 inch), and through a transition zone consisting of several layers of filtering material. Each layer of filtering material must decrease in size, with the top filter layer being composed of the finest sand (all layers except the top layer are put in place with a front end loader, while the top layer is placed either pneumatically through the fence or hydraulically through the fill pipes as the first pour of backfill material); the top layer must extend to a depth of at least three feet above the brow of the drawpoint. Specifications are given in Soderberg and Busch (1985) for selecting a proper grain size and filter material for the plug.

The fill fence design suggested by the U.S. Bureau of Mines for use with its ideal plug consists of a reinforced concrete base which extends to two feet above the top of the drain pipes. Horizontally laid square timbers supported by vertical posts installed above the concrete portion of the fence complete the structure. Spaces are left between the timbers to allow free seepage of water. In actual use, water should never accumulate to a level above the concrete portion of the fill fence. If water is seen draining through the horizontal timber section, it is an indication that the drain plug is not functioning properly and filling operations should be suspended until the problem is rectified (Soderberg & Busch 1985).

As can be seen from figure 4. 13 this plug design is a little too complicated for normal use in an operating mine. It requires a reasonable degree of engineering control to make sure it is placed correctly. The cemented containment plug might cost more to place because of the cement, but it is easier to manage in an operating environment.

4.5.4 Discussion of Containment Approaches

The most secure approach to fill containment is the construction of a massive bulkhead with auxiliary drainage for extra security. Unfortunately this is also a high cost labour intensive solution to the problem. The alternatives all involve some risk which should be understood, but which can be accounted for in the engineering design.

A containment plug is dependent upon the long stability of the cement bonds to maintain cohesion. If the fill is a tailing which contains sulfide material, oxidation of the sulfide may result in the gradual breakdown of these cement bonds. This could lead to a potentially dangerous situation. The degree of fill oxidation observed in most fills is marginal and limited to long term exposed faces, but this does not mean it can not happen. Installing auxiliary drainage with a containment plug will serve as additional insurance against the danger of liquefaction.

The use of free draining designs requires confidence in the operations ability to place sufficiently permeable fill in the drift plug section and stope bottom. This requires thorough knowledge of the fill properties, maintaining good quality control and meticulous monitoring of the pour, conditions not often fully achieved in practice. A further consideration is the long term performance of the bulkhead and fill. Drainage characteristics of fill tend to deteriorate with time as consolidation affects the percolation paths. If water continues to enter the stope from natural ground sources or mining above, the fill may become saturated and the pressures on the bulkhead may be excessive.

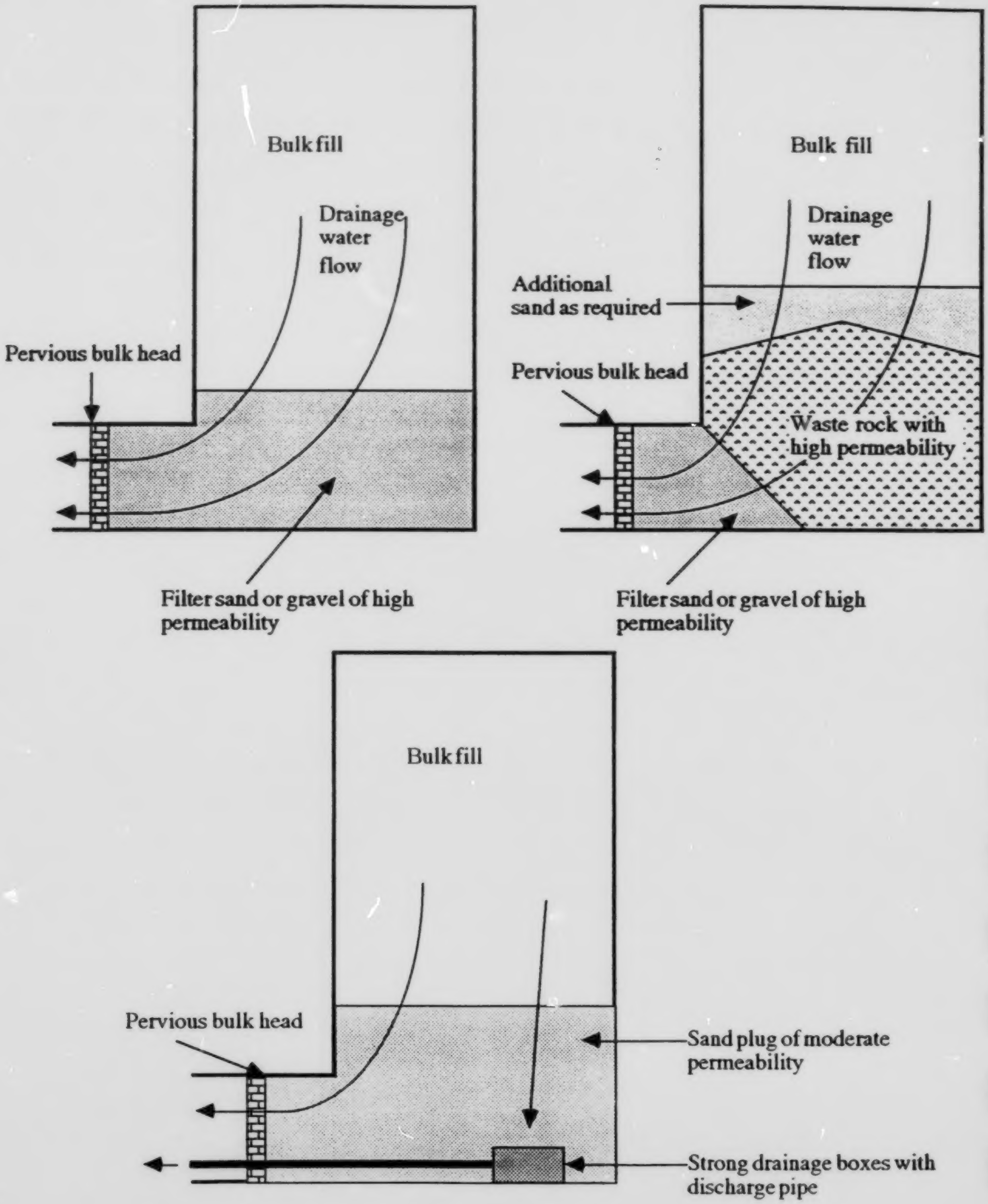

Bulk fill
Drainage water flow
Pervious bulk head
Filter sand or gravel of high permeability
Bulk fill
Drainage water flow
Additional sand as required
Pervious bulk head
Waste rock with high permeability
Filter sand or gravel of high permeability
Bulk fill
Pervious bulk head
Sand plug of moderate permeability
Strong drainage boxes with discharge pipe

Figure 4.12, Drainage blanket fill containment concepts. (after Smith, Mitchell, 1982)

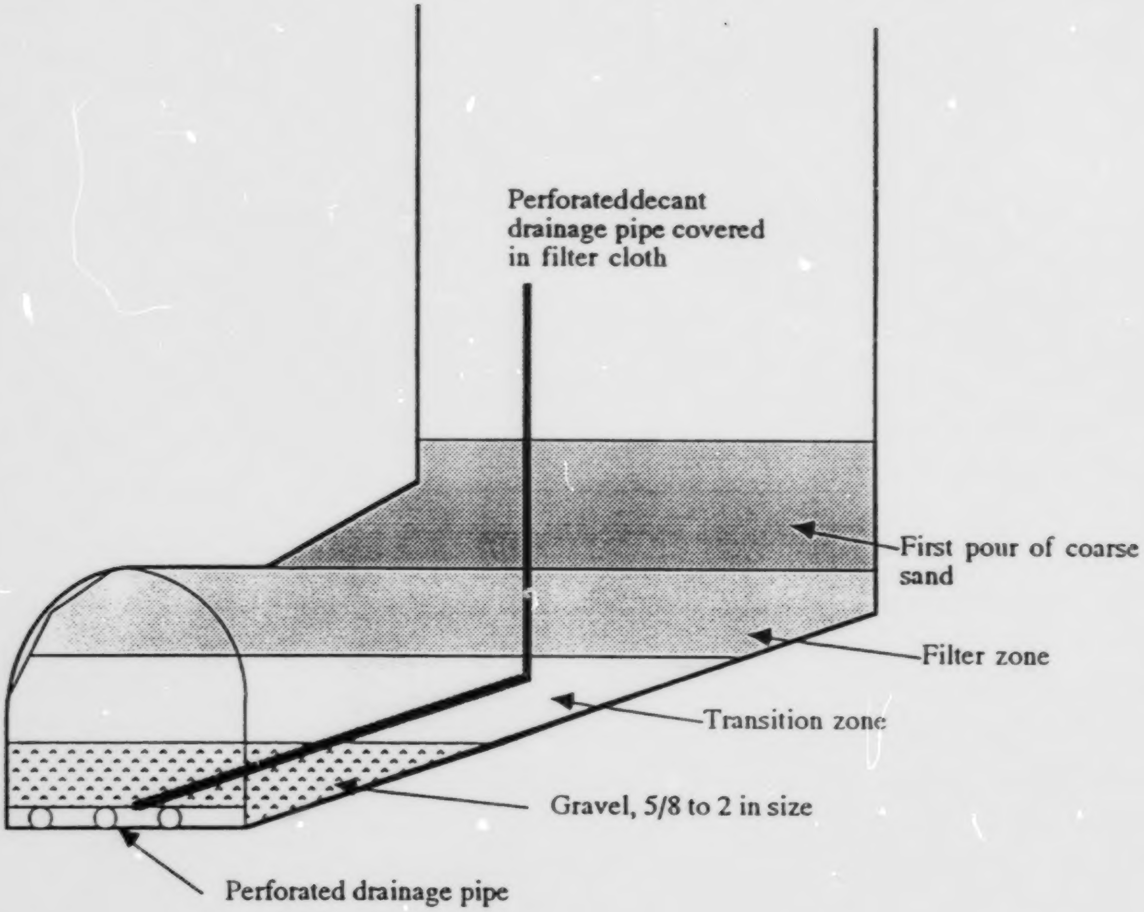

Perforateddecant drainage pipe covered in filter cloth
First pour of coarse sand
Filter zone
Transition zone
Gravel, 5/8 to 2 in size
Perforated drainage pipe

Figure 4.13, USBM fill containment drainage blanket design. (after Soderberg, Busch, 1985)

4.5.5 Fill Fence Performance and Water Pressure Monitoring

Monitoring of hydraulically backfilled stopes is important to ensure that fill plugs and fences are functioning properly. It can be done with instrumentation (pressure meters etc) or by simple water balance measurements. Instrumentation might be justified for large or critical pours but it is more common just to monitor with water balances. A water balance involves measurement of the following:

1. Natural seepage through the drift opening prior to filling.

2. Daily delivery water to the stope (calculated from sampling the pulp density of the backfill being delivered)

3. Daily quantity of water draining through at all of the fill fences for a given stope.

Water balance measurements give a good indication of the degree of fill saturation, which should be maintained below 60 percent in a well drained stope (below 50 percent for high sulphur content tailings fill). If saturation exceeds 75 percent, the system should be reviewed to determine possible dangers (Smith & Mitchell 1982). These measurements should be continued after completion of drainage for each pour, especially if uncemented backfill is used. This allows prediction of the danger of re-saturation or liquefaction of the fill which could be triggered by adjacent blasting activities or by inflows of water from sources outside of the filled stope.

The determination of pressure against a fill fence is usually done by accurate measurement of the deflection of the fence itself. It is stated that if the deflection of a timber fence is measured to the nearest 0.1 mm, the pressure on the fill fence can be calculated to the nearest 1.3 kilopascals. Details of the technique are given in Smith & Mitchell (1982).

4.6 PREPARATION

The preparation of hydraulic fill is a comparatively simple operation which can be done with the minimum of equipment. However as the demand for quality and consistency increases so does the complexity of the preparation plant. When the production of consolidated fill is required the sophistication required jumps dramatically. The basic processes stay the same but instrumentation and control become more critical.

A detailed discussion of hydraulic fill preparation is out side the scope of this report. To do it justice would take considerable effort. However a short review is included and a couple of standard flow sheets included for the sake of completeness.

A typical hydraulic fill preparation plant will consist of three basic components; material receiving, material storage, and material proportioning, mixing and pouring. In addition sizing of the fill material, if required, may be included as part of the preparation plant and will be carried out first.

Hydraulic fill material, particularly mill tailings, may need sizing before being placed. Hydrocyclones are the most common piece of equipment used for sizing hydraulic fill materials. They have a number of advantages; high capacity, dewater as well as size, require little space, are reasonably efficient, and have moderate capital and operating cost. They do not produce a perfect size cut but perform adequately for the requirements of hydraulic fill. The required size cut and recovery can be achieved by placing cyclones in circuit; series or parallel or a combination of both.

Binders are stored in separate silos. They are added to the fill in agitated mixing tanks just prior to its being discharged to the delivery line. The accurate weighing and control of binder addition is critical to achieving the design consolidated fill mix and requires considerable instrumentation of the whole preparation process.

The most important design parameter for a fill preparation plant is to ensure a constant high pulp density. This is achieved in a variety of ways, usually involving some degree of instrumentation control.

The following collection of hydraulic fill plant flow charts illustrates the preparation process.

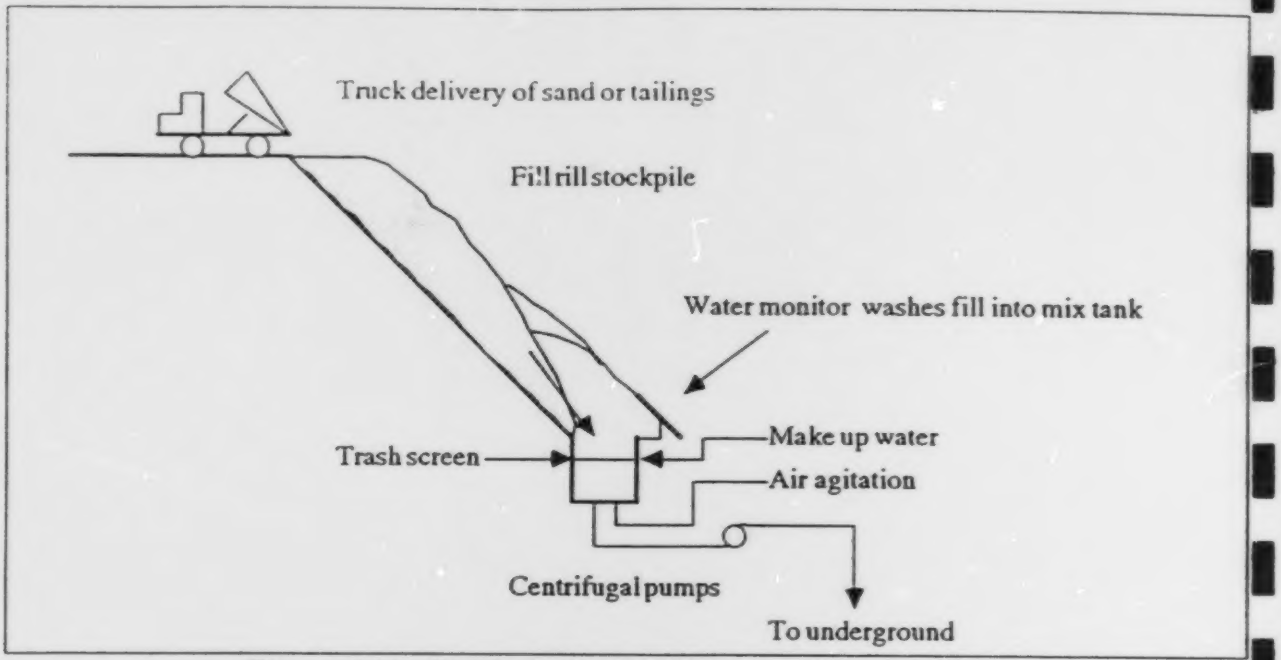

Truck delivery of sand or tailings
Fill rill stockpile
Water monitor washes fill into mix tank
Make up water
Trash screen
Air agitation
Centrifugal pumps
To underground

Figure 4.14, Simple hydraulic fill preparation plant.

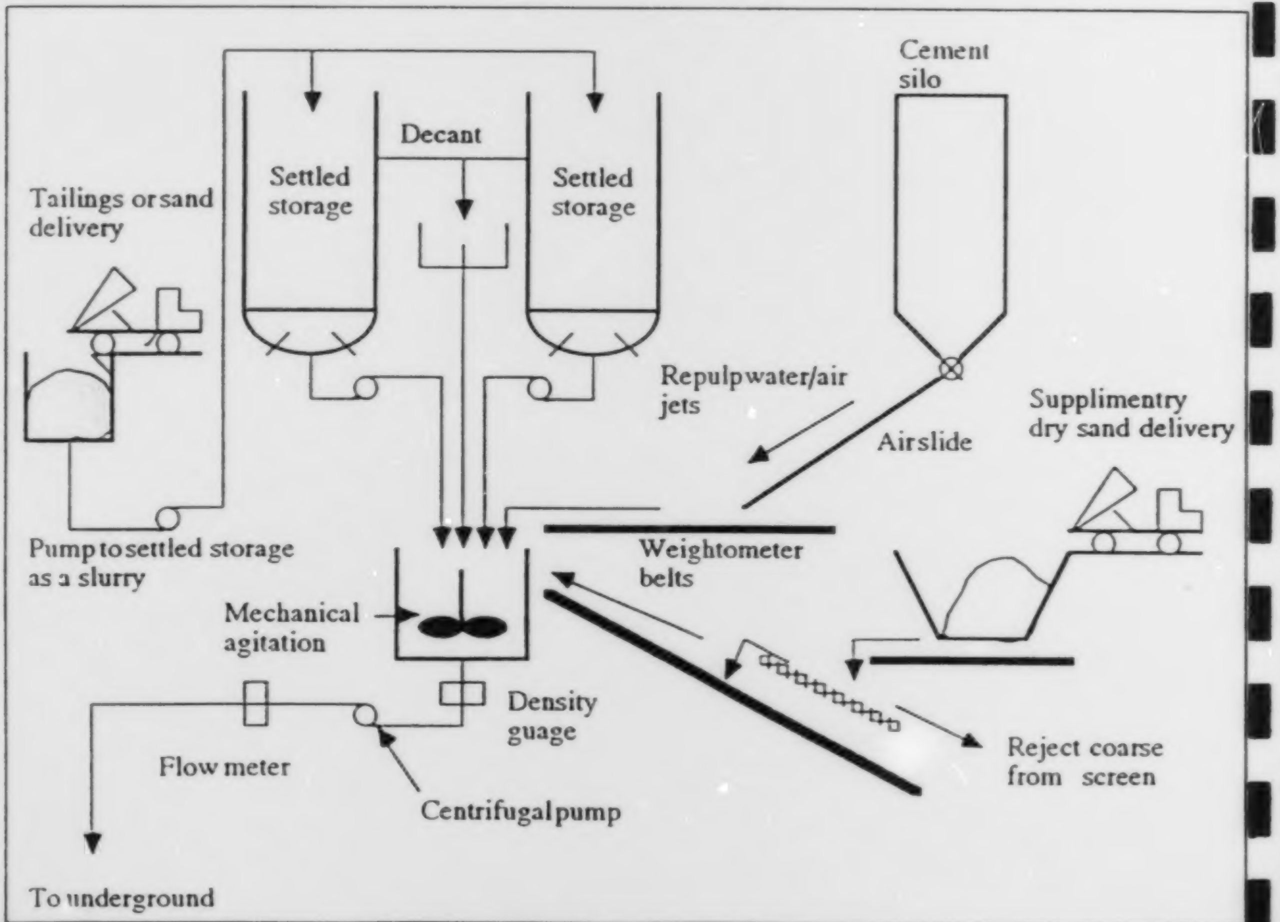

Cement silo
Decant
Settled storage
Settled storage
Tailings or sand delivery
Repulp water/air jets
Supplimentry dry sand delivery
Airslide
Pump to settled storage as a slurry
Weightometer belts
Mechanical agitation
Density guage
Reject coarse from screen
Flow meter
Centrifugal pump
To underground

Figure 4.15, Hydraulic fill preparation plant.

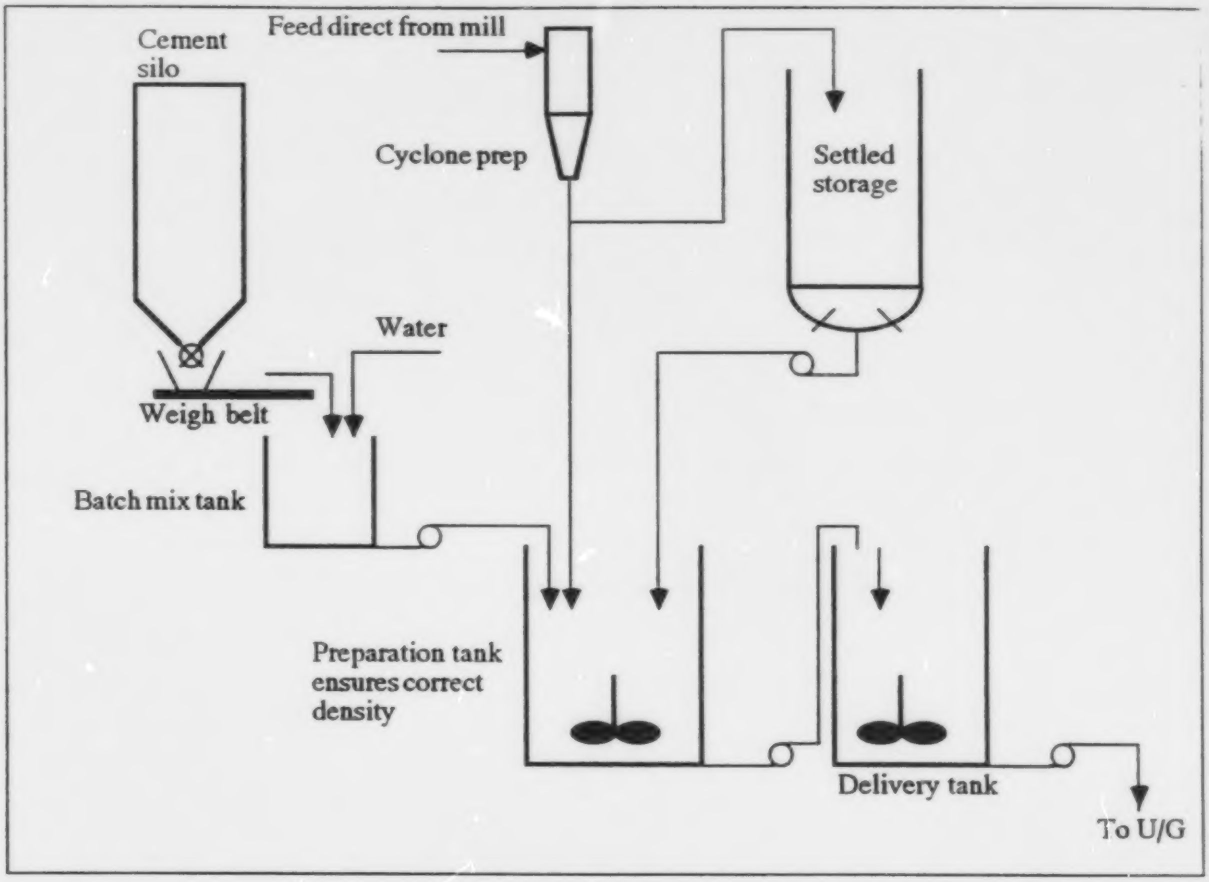

Cement silo
Feed direct from mill
Cyclone prep
Settled storage
Water
Weigh belt
Batch mix tank
Preparation tank ensures correct density
Delivery tank
To U/G

Figure 4.16, Hydraulic fill preparation plant.

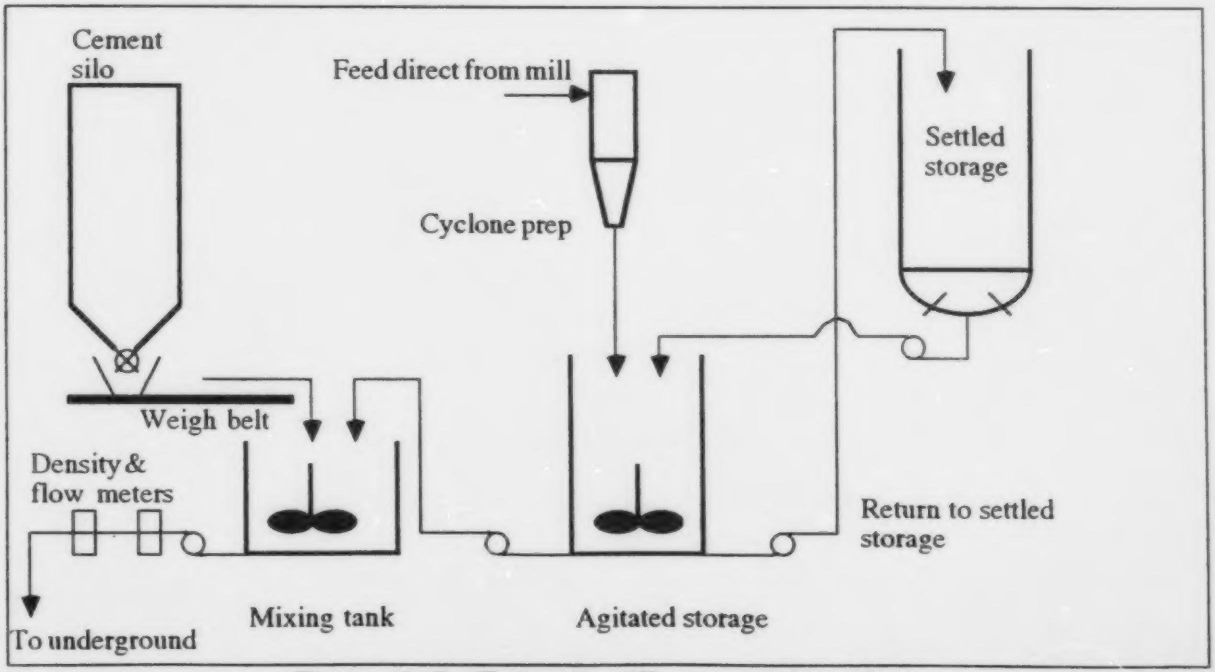

Cement silo
Feed direct from mill
Cyclone prep
Settled storage
Weigh belt
Density & flow meters
Return to settled storage
To underground
Mixing tank
Agitated storage

Figure 4.17, Hydraulic fill preparation plant.

4.7 RESEARCH AND DEVELOPMENT

The success of any hydraulic backfilling operation depends mostly on the proper selection of backfill materials, binders and additives, and on the optimal proportioning of each of the backfill components. These decisions must be based on a complete array of laboratory, field and in-situ tests. Factors such as granulometry, percolation rate, liquefaction characteristics, densities, porosity, void ratio, transportation characteristics, attrition of particles, pyrite and pyrrhotite contents and structural components such as shear resistance, cohesion, friction angle, uniaxial compressive strength, triaxial strength, modulus of deformation and the self-heating properties should be determined to assess any material's suitability as backfill (Nantel 1983). The laboratory procedures for determining these properties were discussed in chapter two and appendix H.

4.7.1 Laboratory Tests

Laboratory tests should be conducted on a wide range of potentially suitable backfill mixes by varying the type of material used as aggregate, the type and quantity of binder used, the setting and curing time of samples, the water type (e.g. tap versus mine water or lake water), the presence or absence of additives such as flocculants, etc. These variables should be incorporated into all laboratory tests conducted on hydraulic backfill material. Once the tests have been completed, an attempt should be made to correlate different parameters. For example, the effect of the mill grind size or of varying cement contents on bulk density, percolation rate and strength should be assessed. And finally, results of laboratory tests should be interpreted with cost efficiency in mind so that the most suitable backfill mix selected is cost effective in practice.

Laboratory strength analyses of backfill mixes is accomplished through conventional uniaxial (unconfined) compressive strength tests or triaxial tests. Triaxial measurements help determine the effect of confining pressure on strength. They also permit the determination of additional parameters such as cohesion and internal angle of friction of the mix. Special confining cells must usually be manufactured to accommodate triaxial tests on relatively large backfill samples. For example, Mt. Isa Mines in Australia has had special cells designed and manufactured to allow triaxial tests to be conducted on backfill samples measuring 760 mm high and 380 mm in diameter.

The most common and simplest strength test for backfill is the unconfined compressive strength test. This test is usually conducted with samples cast in cylindrical forms, although there are many reports of strength tests done on backfill cubes. Enough specimens must be tested to be statistically representative.

This project has adopted a minimum sample size of three as being representative.

Conventional cemented backfill achieves a value very near its ultimate strength over a curing period of 28 days. Most strength tests on cemented backfill are conducted at that age. If binders other than cement are being tested, it is good practice to carry out strength tests at several curing times varying between one and 28 days. This is because several pozzolans develop their strength at a different rate compared to conventional portland cement. Testing of portland cement against alternative binders for this project compared strengths up to 56 days of curing.

When curing, the samples should be placed inside the mine so that the full effect of the ambient conditions on the curing process is obtained. Care must also be taken to reproduce as closely as possible the true backfill mix which would be produced at the plant. This means using mine water to mix the sample batch rather than tap water if that is what will be used at the plant. Mixing times should also represent as closely as possible those which will be in effect during fill plant operation. If the plant is already an operating one, backfill samples for strength tests should be obtained directly from the point of pouring.

There are many factors affecting strength of backfill mixes and therefore, all variables such as moisture content, void ratio, material type, binder type and quantity, curing time, water type, etc. should be related to strength. And finally, if possible, samples cored from in-situ fill after pouring and curing should be tested and compared with laboratory sample to determine any discrepancy between laboratory and actual field values. Care must be taken in producing cored samples in-situ so that effects of vibration, heat, cooling water and general handling on sample authenticity are minimized. To this effect, Falconbridge Ltd. and CANMET have developed an advanced coring technique for backfill to produce core samples which are undisturbed and are as close to true in-situ backfill as possible. This is described in a previous project report, In situ Monitoring and Computer Modelling of a Cemented Sill Mat and Confines during Tertiary Stage Pillar Recovery.

CHAPTER 5

ROCK FILL

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5.1 INTRODUCION

The strength and stiffness which can be achieved with consolidated rockfill have encouraged its increasing use as a backfill. There are strong economic links between the strength of backfill, the stable fill exposure dimensions, the possible stope dimensions and hence the profitability of a mining operation. Consolidated rock fill has the capacity to stand over exposures not economically possible with hydraulic fill. The stiffness which can be achieved with rock fill offers some significant advantages to operations seeking to control ground subject to high stresses, control which is not possible with hydraulic fills. Rockfilling also provides an inexpensive method of disposing of development waste underground.

Unconsolidated rockfill was the first backfill to be used. It is still common, normally used as a void filler for providing passive support, by minimizing regional ground movement problems. It has a limited ground support and negligible free standing height capability, (limited to the natural angle of repose which is around 35 degrees for dry fill. It is commonly used when the fill is not to be exposed by future mining activity. This project has been mostly concerned with consolidated rockfill.

Kidd Creek Mines, a major user of rock fill, is recognized as being expert in the technology of its application. They were a principal subcontractor to this project and carried out extensive test work into all aspects of rock fill. Much of this chapter considers practices at the Kidd Creek Mines operations. The emphasis in this chapter is on rockfilling large blasthole stopes, which is the mining method at Kidd Creek.

5.2 FILL TYPES

Consolidated rockfill is comprised of sized or unsized aggregate mixed with various types and amounts of binder. It yields a higher strength fill with less binder addition compared to cemented hydraulic fill. With equivalent binder contents, consolidated rockfill will exhibit uniaxial compressive strengths two to three times higher than those of hydraulic fill. It has a higher modulus of elasticity, cohesion and angle of friction compared to hydraulic fill, but it is also more difficult and expensive to place.

Rock fill may be concurrently or post consolidated. Post consolidation of rockfill involves filling the stope with coarse aggregate and subsequently percolating a cemented hydraulic fill into this mass. For example at the GECO Mine, rock filling starts when ore mucking opens a void and with gradual mucking of ore, more rock waste is introduced on top to keep the stope full of material. Once the ore has been completely mucked, the stope is full of waste and then consolidated 30:1 tailings fill is introduced from the top to consolidate the waste. This method requires a very coarse uniformly sized waste rock to ensure adequate percolation and binder coverage of the aggregate. Falconbridge has conducted several test programs investigating the potential for post consolidation of mine development waste as rockfill. The results were very discouraging, percolation of the binder slurry was erratic and resulted in a very weak backfill mass even while individual parts of the mass were very strongly cemented.

Placement of concurrently consolidated rock fill with the binder already added is more normal. A binder slurry will be sprayed onto the rock as it leaves a conveyor entering the stope, onto rock in the bucket of a loader or tray of a truck, or the binder and rock may be mixed in a bowl and then placed in the stope.

The post consolidation method is suitable when a high degree of segregation has occurred, and where by consolidating the coarse particles at the walls of the stope, ore dilution can be minimized. The concurrent method results in a much more competent and uniformly distributed backfill. The use of the second method at Kid Creek Mines has allowed the mine to drift into and drive raises through consolidated rockfill (Wittchen et al., 1989).

Rock for rockfill is usually sourced from the waste piles of open pit operations, e.g at Kidd Creek Mines and the Williams Mine, or from waste quarries, e.g Mount Isa Mines, GECO and Golden Giant Mines. Underground development and other waste is also commonly used for rockfill when available.

A number of rockfill variations are recognized. The following definitions are used at Kidd Creek Mines Limited;

1. Consolidated rockfill (CRF). Sized rockfill aggregate is mixed with binder slurry at a pulp density of 50-60%, with a binder addition rate usually 5 to 6% by weight of aggregate . This type of fill exerts an active pressure on the contacted wall, providing not only ground support but also improvement of inherent strength of the walls e.g., at KCM and Williams Mine.

   The advantages include; no drainage problems and if placed correctly high quality fill is achieved.

   The disadvantages are; segregation control is difficult, quality can be variable, access and layout is critical.

2. Consolidated sand rockfill (CSRF). This is a combination of CRF with varying amounts of

sand added, usually 5-10% of the CRF weight. With the same overall cement content as the CRF, a cement sand slurry is introduced simultaneously with the coarse aggregate and cement slurry. The sand will fill the voids in any segregated aggregate. This enhances the fill stability for both gravity loading as well as blasting vibration resistance during excavation of adjacent stope or pillars.

The advantages are; raise layout is not critical, it has relatively good mobility and less segregation than CRF, it has a lower angle of repose than CRF, and it is denser than CRF.

The disadvantages are; relatively good access is required, there is a slurry runoff control problem.

3. Consolidated sand waste fill (CSWF). Waste is left in place and consolidated by percolating a cement sand slurry mixture through it. The cement by weight of sand is around 18%, but overall cement content is around 5% by weight of sand and rock. Pulp density of the sand slurry is 65 to 70%. Pulp density for a straight cement grout is 55 to 60%. The richer cement content of this fill will consolidate the waste to improve ground support and reduce dilution.

   Its advantages are; good access is not essential, it saves having to remove waste, specific areas of a stope such as individual walls can be consolidated.

   Its disadvantages are; bulkhead control is essential, the path of the sand slurry flow is difficult to control, and poor results are common as waste is not very permeable.

4. Consolidated sand fill (CSF). Consolidated sand fill is a cement-sand slurry mix, with a cement content, 5 to 10%. The cement sand slurry is placed after the majority of the stope has been filled with CRF in order to tightly fill the remaining void beneath the stope back, thus providing roof support.

   Its advantages are; it is very mobile, has a low angle of repose, good access is not essential.

   Its disadvantages are; bulkhead control is essential, there is a slurry runoff control problem.

5. Unconsolidated rockfill (URF). Unconsolidated sized or unsized waste from surface or underground placed as an inexpensive fill, but which can only be regarded as applying a

passive pressure on stope walls and has limited effect on supporting ground.

Its advantages are; it is cheap, quick and simple.

Its disadvantages are; it is not consolidated and offers limited ground support.

5.3 DESIGN

Fill must be able to sustain not only the gravitational loading of the overlying fill material, but also the dynamic effects applied during blasting. In designing a fill it is necessary to consider both the static and dynamic fill strength. The design process is not an easy one, but requires considerable research work. For example, after many years of investigation in the search for a suitable backfill, Kidd Creek Mines decided upon a strong consolidated fill formed from mixing crushed aggregate from pit waste with a ratio of 20 parts aggregate to 1 part cementing agent. It was found that this produced a stable fill mass which would reduce dilution and the risk of ore loss during the mining of pillars. The aggregate cement ratio in other rockfill operations ranges between 30:1 and 10:1.

5.3.1 Static Strength

Uniaxial compressive strength, UCS, is one of the most important parameters to be considered when dealing with consolidated backfill. The uniaxial compressive strength requirement of the consolidated rockfill should be determined by defining the height and width of the most likely fill exposure. The uniaxial compressive strength in different rockfill operations ranges between 2.3 to 7 MPa. This compares to strengths up to about 2 MPa for hydraulic fill.

For example at Kidd Creek Mines (KCM), it was anticipated that the exposed fill face would have a height of 120 m and length of 60 m. To support the gravity loading alone, the fill required a compressive strength of 2.8 MPa. A safety factor of 2.5 was applied to allow for additional blast loading and the reduction of fill strength if inadequate mixing occurs. Thus a fill having a strength of 7 MPa, (as measured in the laboratory) was designed.

Figure 5.1 shows the relationship between the compressive strength and the cement content for 15 cm cylinders of consolidated rockfill at Kidd Creek Mines. The compressive strength of these cylinders at 28 day curing time is related to the cement content by the following formula.

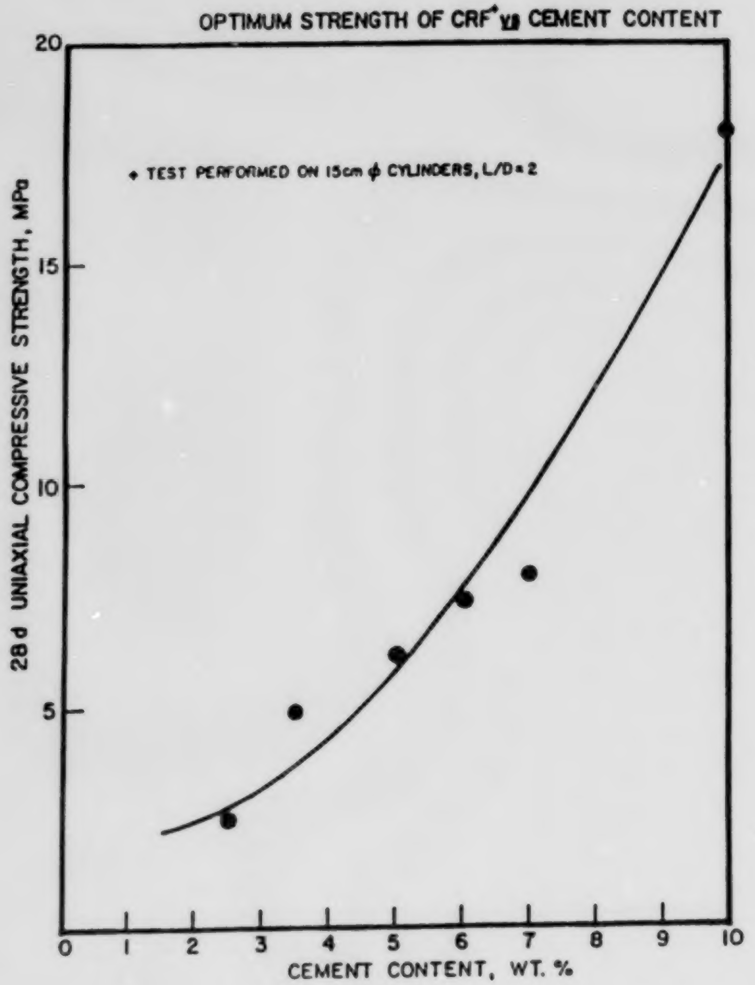


Figure 5.1, Relationship between unconfined compressive strength and cement content for KCM rockfill.

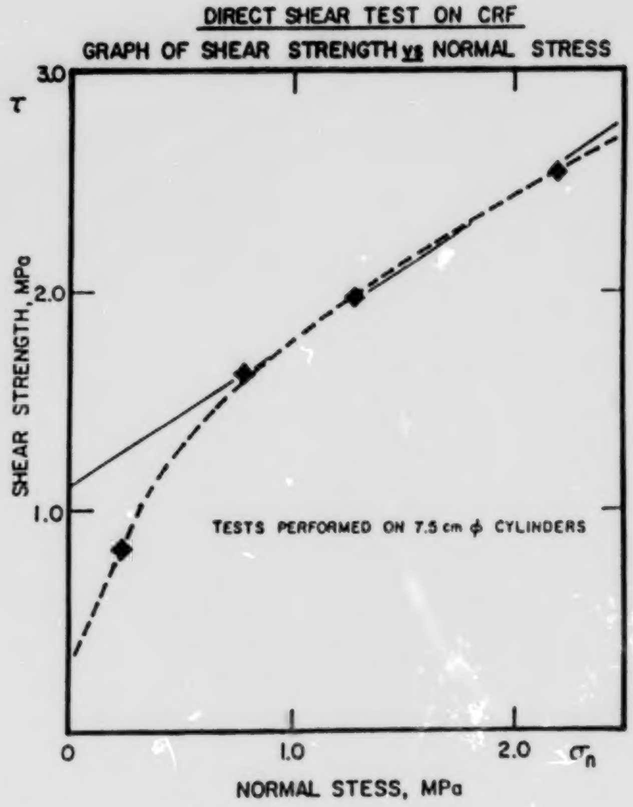


Figure 5.2, Results of direct shear tests on KCM rockfill.

At Kidd Creek Mines;

$$UCS = 1.5e^{0.25C} \qquad \text{for } 2 <= C <= 10$$

where:

UCS = Uniaxial Compressive Strength of the rockfill, MPa

C = Portland Cement content by weight %, of minus 4cm aggregate.

The shear failure mechanism by gravity loading on backfill has been discussed elsewhere (Askew et al., 1978) and in Chapter 2. It can be expressed by the following equation using a wedge shear sliding model.

$$Fs = (\text{Shear Strength}) / (\text{Driving Force})$$

$$Fs = (cL + N \tan\emptyset) / W \sin D$$

where:

Fs = Static factor of safety

c = apparent cohesion

L = length of the sliding plane

Ø = apparent friction angle of the plane

N = effective normal reaction of the plane

W = weight of the sliding block per unit width

D = dip of the sliding plane.

Cohesion and friction angle values of rockfill should be evaluated for stability analysis. The shear strength of cemented rock fill can be determined using the direct shear test. The results for Kidd Creek rockfill, (see Figure 5.2) show the relationship between the shear strength and the normal stress. As measured from the graph, the apparent cohesion and the apparent friction angle are 1.1 MPa and 33 degrees respectively. This corresponds well with the observed angle of repose of the fill, normally 34 to 37 degrees. Cement addition to the fill mix increases the shear strength, apparent cohesion, and the modulus of elasticity.

5.3.2 Dynamic Strength

The parameters governing blast damage to consolidated fill are not fully understood yet. It is believed that the blasting vibration resistance may be closely related to the dynamic tensile strength of the fill.

The dynamic strength required in a rockfill can be related to the blasting vibration level in terms of peak particle velocity as shown in the following equation.

$$Fd = (\text{Dynamic Strength})/(\text{Dynamic Loading})$$

$$Fd = T/\ dPV$$

Where:
Fd = Dynamic factor of safety
T = dynamic tensile strength, say 5% of UCS
d = fill density
P = fill compressional wave velocity
V = particle velocity at failure in fill.

Consolidated rockfill being similar to weak concrete can be strong in compression, yet weak in tension especially in the outer zone of the fill pile, or adjacent to stope walls where most of the segregated aggregates exist. The weak zone should be reinforced to enhance the tensile strength if the degree of blast damage is to be minimized.

To minimize the effect of segregation tests were conducted at Kidd Creek Mines into the effect of adding sand into consolidated rockfill. The sand was intended to act as a void filler. The results indicated that the addition of sand significantly increased not only the compressive strength at a particular 5% sand content, but also the dynamic strength of the rockfill. Figures 5.3 and 5.4 show theses results. This material is known as consolidated sand rockfill (CSRF), as defined previously.

Another reason for the use of the consolidated sand rockfill is that the need to coat all aggregate with cement slurry becomes less critical. This implies that a lesser degree of mixing of cement slurry with the aggregate is acceptable. Hence the need to provide a number of fill raises in order to reduce segregation becomes less critical (Yu and Counter, 1983). The use of consolidated sand rockfill has been limited to selected stopes where a more stable fill exposure is required.

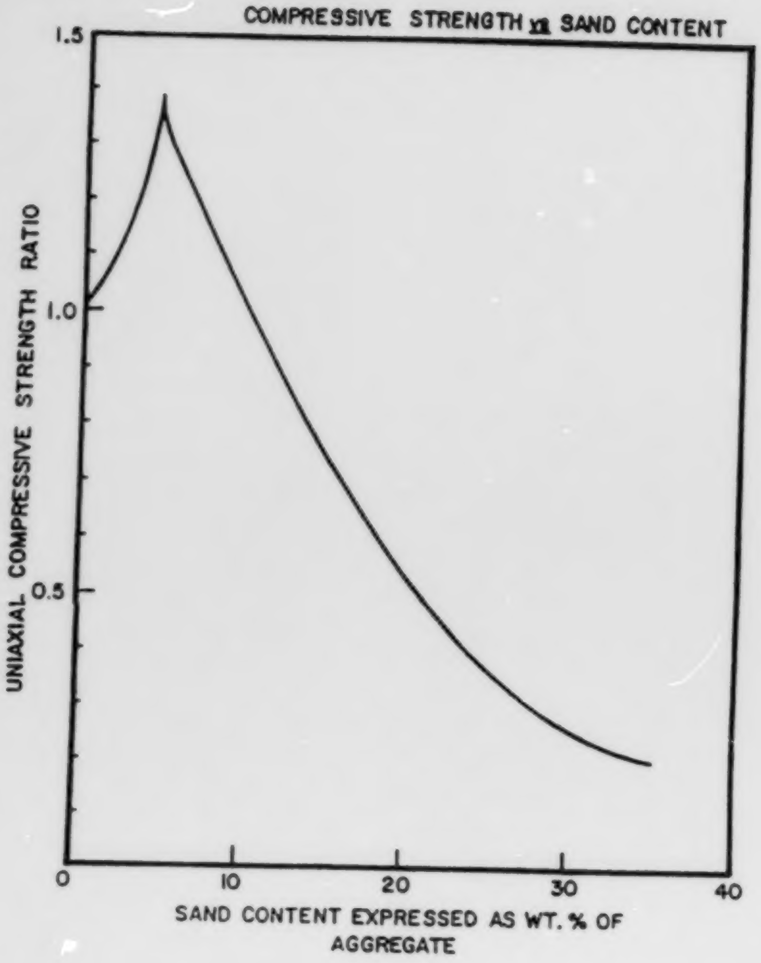


Figure 5.3, Unconfined compressive strength versus sand addition for KCM rockfill.

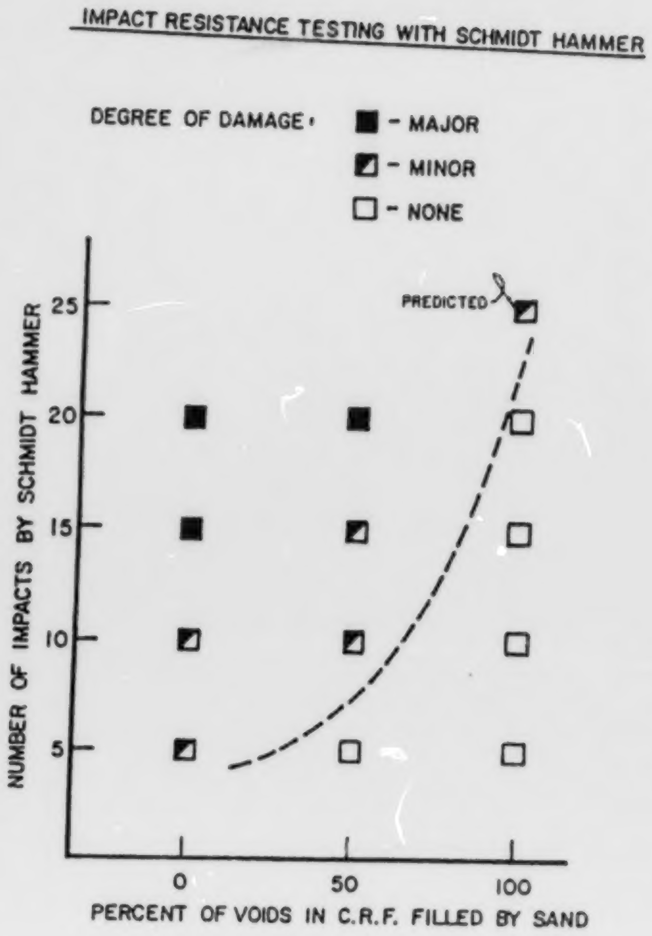


Figure 5.4, Impact resistance of KCM rockfill at various sand addition rates.

5.3.3 Physical Properties

Rockfill is not a uniform material. Due to the segregation effect, a large range of in-situ densities exists in a cemented rockfill mass. Typical in-situ bulk densities are 10-20% lower than those measured in the lab. Rockfill has a moisture content ranging from 2 to 5% (by weight) with an average porosity of 34%. The compressive strength of in-situ rockfill varies substantially from 1.3 MPa to 11 MPa.

The measured physical properties of consolidated rockfill at Kidd Creek Mines are listed below, (average values where appropriate).

Bulk Density	1.88 tonne/m^3
Void Ratio	0.51
Moisture Content	3.5 %
Compressive Strength	4.8 - 7 MPa
Elastic Modulus	3.4 GPa
Poisson's Ratio	0.35
Friction Angle	37 deg.
Cohesion	1.1 MPa
P-Wave Velocity	3,150 m/sec

Figure 5.5 shows the particle size analysis for typical Kidd Creek Mines rockfill aggregate. This is waste rock from an open pit mining operation which is crushed and screened in a surface preparation plant.

The test work was conducted using 6" diameter by 12" high test cylinders.

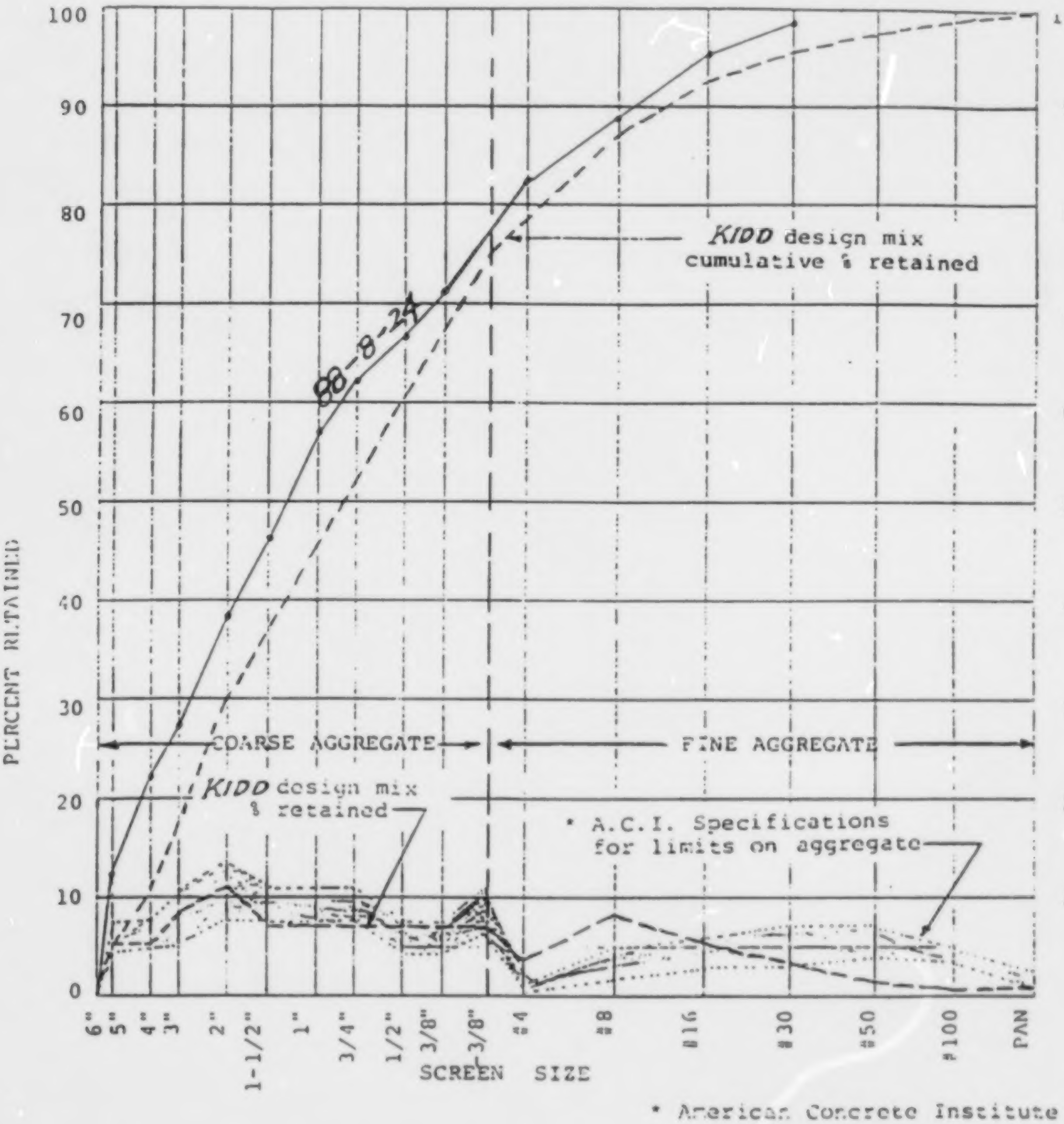


Figure 5.5 Typical size distribution of fill aggregate at Kidd Creek.

5.4 PREPARATION

Rockfill aggregate supplied from surface will usually be subject to some preparation prior to being sent underground. Underground waste used for rockfill will commonly be used as is.

5.4.1 Aggregate and Sand

The production and handling of various aggregates for rockfill needs rugged equipment due to the volume and size of the materials. Rockfill material is transferred underground from surface by dumping from a front-end loader, truck or conveyor into raises which are commonly between 1 and 2.4 metres in diameter. The coarse and fine aggregates are usually sized and mixed on surface before going underground. This allows the removal or addition of different size fractions to meet individual stope requirements.

For example at Mount Isa Mines, Australia, the waste rock is quarried, crushed and screened to produce up to 1300 tonne per hour of minus 300 mm material with 5% passing 25 mm. Screen product is conveyed 2500 m along surface, drawn 550 m down a 2.4 m diameter raisebored pass and distributed by a multi-conveyor system into the desired stope, (Thomas et al., 1979). At Kidd Creek Mines preparation of fill material is accomplished at the surface backfill plant by crushing waste rock to a maximum size of 15 cm.

5.4.2 Binder

Binder for rockfill is commonly transported separately from the aggregate and the two components are combined close to the point of placement. The binder will usually be prepared as slurry which is then sprayed on the aggregate. The slurry may be prepared on surface or at an underground preparation facility. Surface preparation requires a network of boreholes and pipes for distribution to underground. For underground preparation, transport of dry binder is required. Dry pneumatic conveying has been used for large tonnage operations and transport by bulk tanker or bag for smaller applications.

Preparation of the binder slurry is usually a batch process to ensure proper proportioning and quality control. If surface preparation is used it is common to have a receiving tank underground where each batch is remixed and kept agitated until it is used. While one batch is being used the next may be prepared on surface.

A binder slurry will be prepared at a solids concentration of from 50% to about 65% by weight. Higher concentrations are used to minimize the water to binder ratio, but lower concentrations may be required to facilitate flow through long pipe lines or for mixing with very dry aggregate.

5.5 PLACEMENT

5.5.1 Aggregate Distribution

Vertical transportation of aggregate is commonly by raise. Horizontal underground transport is generally by conveyor, truck or scooptram. Transportation with scooptram or truck is the highest operating cost method for fill handling. It is employed in the cases where the drift size does not allow the installation of a conveyor system, or the tonnages involved do not justify the capital expense of a conveyor system. For example at Williams Mine, 40,000 tonnes of fill or less will result in using the scoop or truck hauling method. For tonnages in excess of this costs favour the use of conveyors. Kidd Creek Mines and Mount Isa Mines, both large tonnage backfill operations use conveyor transport for aggregate distribution.

5.5.2 Aggregate Attrition

The two main areas of concern with regards to surface supplied aggregate used in consolidated rockfill are attrition and moisture content.

Attrition is the mechanical breakdown of the aggregate particles as they are handled in the transport system. Experience has shown that aggregate suffers greatly as the depth to which it is transported increases, (Yu, 1989). Attrition leads to an increase in the fines content of the aggregate. Additional binder is required to coat the extra fines to achieve the designed fill strength. The result of improperly sized aggregate, due to attrition can be very costly. For example each increment of 1% binder used at Kidd Creek Mines increases the annual binder cost by $2,000,000.

The attrition of aggregate in a fill raise was observed at Kidd Creek Mines. The results, expressed in terms of the attrition ratio, A, are described by the following equation.

$$A = (1+h)/1100$$

Where:

A = (D_{50} at Surface):(D_{50} at h), the attrition ratio

D_{50} = Aggregate size at 50% passing

h = aggregate vertical travel distance (m)

The results are also shown graphically in figure 5.6, (Bronkhorst, 1986).

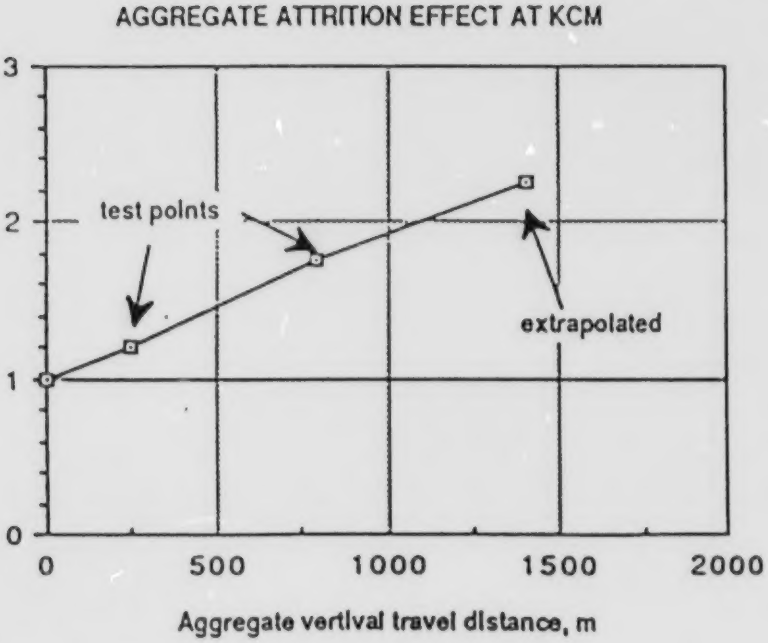


Figure 5.6 Aggregate attrition at KCM. (After Bronkhurst, 1986)

By using the graph or the equation, the relative size of aggregate in terms of D_{50} at any depth can be found. For example, the attrition ratio at h = 900 m is 1.8. If the D_{50} of aggregate was 8 cm at surface, the D_{50} of degraded size after passing through a 900 m raise would be expected to be 4.5 cm (Yu,1989).

If the aggregate is sent underground wet or becomes wet on route, the fines will tend to coat the coarser particles, preventing the bonding between the aggregate particles and cement paste. Wet aggregate is caused by one or more of the following situations.

1. Ice crystals or snow mixed with the aggregate.
2. The temperature of the air occupying the voids passes its dew point, resulting in condensation on the aggregate.
3. Seepage into backfill raises.

5.5.3 Segregation

When a stope is filled by pouring from above, segregation of the rockfill is unavoidable. The flow of fill down any slope is subject to differential settling. The degree of segregation is governed by the pour point orientation and the geometry of the opening to be filled. It will differ for each stope, as stopes are neither identical nor possess the same backfill pour point orientations. It is found that a zone of fine aggregate tends to occur near the impact area, whilst the larger aggregate particles tend to the outer edges of the rockfill pile. The central impact zone tends to consume most of the cement paste, leaving a low cement content rockfill at the perimeter of the fill cone.

The segregation phenomenon becomes more pronounced when stopes are filled by conveyors, due to the impact velocity imparted by the speed of the belt. When a stope is filled by mobile vehicles, only the largest particles have the momentum to travel to the further stope wall. The rest of the material fill the stope by progressive slumping resulting in a more uniform product.

The segregation effect is not all bad, it may be used to give some structural design capacity as discussed in section 5.8.

5.5.4 Backfill Raise

When fill is dumped into a stope via raise, the raise constitutes an important factor to be considered. It should be strategically located and oriented so that a uniform distribution of fill material is achieved. The orientation of the raise determines the location of the fill cone in the stope. When material is in the fill raise, it attains a specific falling velocity which governs the trajectory of the fill into the stope. The trajectory of the fill into the stope can be predicted using the motion of projectile, see Figure 5.7

The material in the raise, however, encounters some frictional resistance, deviating from a freely falling body. It has been observed that in determining the initial velocity leaving the raise, a constant (K=0.42 for Kidd Creek) should be incorporated in the calculation of the falling velocity in the raise. The equations are illustrated below, together with an example of how to compute the horizontal trajectory of the fill in a stope.

For a given dip angle and length of fill raise above a stope to be filled, the horizontal trajectory, X_h, of the fill material can be found from the following equations (Yu, 1989).

$$X_h = V_h \times T$$

$$\begin{aligned} V_h &= V_v / \tan D \\ &= (2g \times L \sin D \times K)^{0.5} / \tan D \end{aligned}$$

$$T = [-V_v \, {}^{+}_{-} \, (V_v^2 + 19.6\ H)^{0.5}] / 9.8$$

where: V_h and V_v = horizontal and vertical velocity components respectively m/sec
L = length of the fill raise, m
D = dip angle of the fill raise, degree
K = 0.42, resistance constant for Kidd Creek Mines
g = gravitational acceleration, 9.8 m/sec^2
T = time taken for the fill material to fall from the raise to the top of the fill cone, sec
H = height of the free fall of aggregate in stope, m

For example, in a fill raise dipping at 60 deg. with an inclined length of 35 m, the horizontal velocity at the end of the raise, V_h, is equal to 9.1 m/sec. If the free falling height is 60 m in the stope, the falling time, T, is 2.24 sec. Thus the horizontal trajectory can be found to be 20.4 m.

A comprehensive study on the number of raises required to minimize segregation at Kidd Creek mines was conducted. It was concluded that the use of two raises in an opening 18 m wide by 45 m long would be satisfactory to limit segregation. Most mines placing rockfill however, utilize only a single fill pour point, as the cost of a second fill raise is high. The larger the stope the greater the incentive to use a second pour point to minimize the rolling distance of the coarser aggregates towards the walls of the stope.

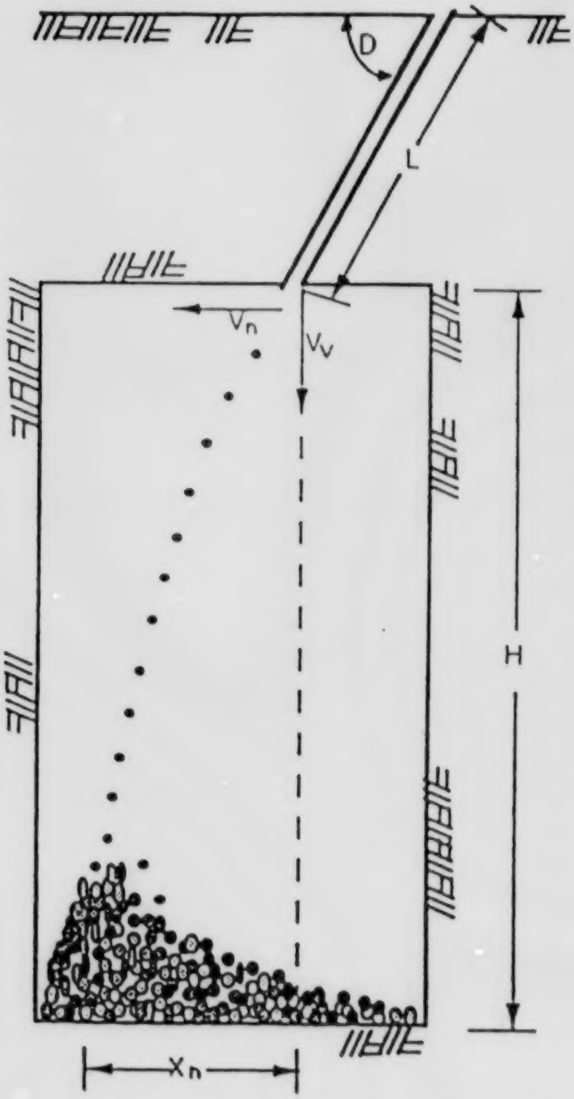


Figure 5.7 Trajectory of particles leaving a fill raise.

5.5.5 Mixing

The key to producing a competent consolidated fill is in thoroughly coating all the aggregate with the cement slurry. If the material is not coated during the mixing process, it may never be properly coated. A number of rockfill mixing arrangements are used. The most effective, but also most complex is the bowl type mixer as used for ready mixed concrete on surface which gives excellent shear mixing. The bowl mixer is a batch system and not suited to conveyor placement methods. It is used by the Brunswick Mining and Smelting company, where a measured quantity of aggregate is bowl mixed with cement in an underground mixing station. The mixed fill is then dumped in a truck for transport to the stope. Kidd Creek No.2 Mine has also used a similar system which was regarded as one of the best methods for providing a well coated fill, but it was discontinued because the demand for backfill exceeded the systems capacity.

When loader or trucks are used for rockfill placement, simple spraying of the binder slurry on top of the rock in the bucket or tray is commonly used. The slurry is given time to mix with the rock during transport and final mixing takes place as the fill is dumped when any surplus slurry gets a chance to percolate through the rock once again. This system is not ideal but gives adequate results with minimum cost for many operations.

Prior to starting the backfill operations in 1976 at Kidd Creek Mines, an extensive study on the mixing problem was made, including full scale testing of a vibratory mixing conveyor, the use of a slusher for mixing, a baffled mixing slide (culvert), and a drum mixer. The simplest system found for conveyor placement to date is based on a baffled slide or chute. The conveyor discharges aggregate on to the inclined baffle, a spray header is mounted over the top of the baffle, and the slurry is pumped from a holding tank spraying the aggregate as it enters the baffle. The tumbling action of the aggregate as it moves through the baffle ensures good coating with binder. The slurry spray helps lubricate the aggregate flow through the baffle. Figure 5.8 shows the arrangement.

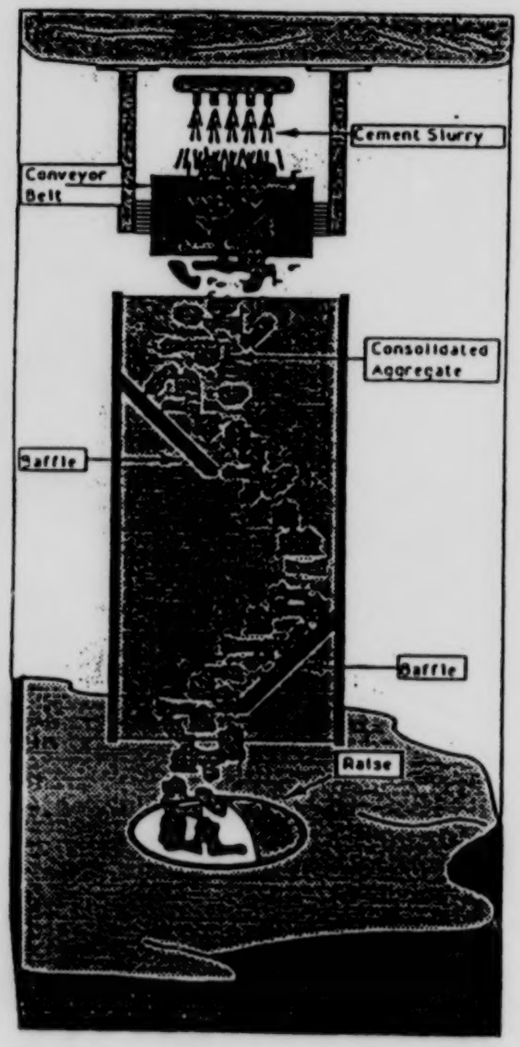
Post consolidated rockfill does not get any aggregate binder mixing and the control and uniformity of this material is consequently low. The following points should be noted about post consolidation.

1. It is impossible to control the flow of slurry in the stope.

2. The slurry will not flow uniformly over the entire backfill cone.

3. The percolation rate of the slurry is variable due to differential settling.

4. Slurry which is not actually used to coat the aggregate acts as a void filler. Since there is insufficient cement to fill all voids, some portion of the fill may remain unconsolidated.



Baffled Slide for Mixing of Aggregate with Cement Slurry to produce Consolidated Rockfill at Kidd Creek Mine

Cement Slurry
Conveyor Belt
Consolidated Aggregate
Baffle
Baffle
Raise

Figure 5.8 Baffle box mixing at KCM.

5.5.6 Impact Damage

This is another important yet easily overlooked factor governing the fill stability. Impact damage can be divided into two categories.

1. When the backfill aggregate collides with the peak of the backfill in the stope, some of the aggregate will break. Should there be insufficient slurry present, the fresh surfaces may never be coated with binder.

2. Should a collision of aggregate and cemented backfill occur after the final curing period, any broken bonds below the placed fill will not be recoated and thus a zone of broken bonds will exist.

To reduce the extent of impact damage, addition of a retarding admixture in the cement slurry is necessary. At Kidd Creek a chemical retarder was introduced at a ratio of 90 cc retarder per 45 kg Portland cement to delay the final set time so that a minimum of 60 cm thick buffer layer of consolidated fill was created. This allowed the shock of dumped fill to be absorbed by the plastic state surface layer preventing impact damage to the cured fill beneath. Another advantage of adding retarding agent to cement slurry was found to significantly reduce the rate of the cement scale build up in the slurry pipelines.

A retarding admixture is not needed if slag or flyash is used as a partial replacement for Portland cement, as the blended binder exhibits a much slower curing rate compared to Portland cement. Kidd Creek who use a flyash and portland blend no longer add a retarder to their binder slurry.

5.6 BINDER ALTERNATIVES

Binder purchase is the most significant operating cost in the placement of consolidated rockfill. Portland cement, the accepted standard binder for rockfill is an expensive product. Considerable savings can be achieved by minimizing the percentage of portland cement used by partially replacing it with cheaper binder materials. Ground blast furnace slag, ground non ferrous slag, and fly ash are the most common replacement binders.

As part of this project Kidd Creek Mines carried out extensive testing of the performance binder alternatives in their consolidated rockfill. Both ground blast furnace slag and flyash have been used as partial portland cement replacements in the Kidd Creek operation. The potential of ground Kidd Creek copper slag as a binder is still under investigation. A discussion of this test work is included as part of Chapter 8, Binder Alternatives.

In considering binder alternatives both the rate of strength development and the ultimate strength are important. Flyash and slag binder blends are characterized by a slower strength development than portland cement, but may achieve a superior ultimate strength. The slow strength development is an advantage in the placement of rockfill where impact damage during placement is reduced with slower setting cement. However the setting rate must be coordinated with the mining schedule, so that the fill has attained sufficient strength by the time that adjacent mining is to take place. This time limit will control the proportion of portland cement which can be replaced by a slower setting alternative. The faster the set required the lower the replacement rate allowed.

5.6.1 Blast Furnace Slag Cements

The long term strength of rockfill using mixtures of portland cement and slag cement as binder have been found to be comparable to that achieved with straight portland cement, at considerably lower cost. Figures 5.9 and 5.10 illustrate the results of test work conducted at Kidd Creek Mines. The effect of slag addition on rate of strength development is shown clearly; slag performs well as a binder provided sufficient setting time is allowed.Test results indicated that at 85 days the strength of consolidated rockfill with 2.5% cement and 2.5% slag by weight as a blended binder (50% replacement by slag cement) attained an equivalent strength of a 5% Portland cement mix (Yu and Counter, 1983).

The addition of 5% sand enhanced the compressive strength of the slag rockfill by 45%. To evaluate the curing process of slag cement rockfill in the underground environment fourteen large test blocks, 60 cm by 60 cm by 100 cm, were cast underground. The dynamic strength characteristics were determined by employing the impact method with a Schmidt hammer and by the detonation of blasting caps within the blocks. This allowed the evaluation of the blasting resistance by the relative size of craters. The tests confirmed the previous observation that blocks containing 5% sand possess a higher dynamic resistance than those with a lower sand content.

A large scale test where a waste pass was filled with rockfill bound with a blend of blast furnace slag and portland cement indicated that no operational problems would be encountered. This test also revealed that the slow curing nature of the slag eliminated the need to add retarding chemicals, resulting in a further cost saving.

After more than two years of testing at Kidd Creek the following conclusions were made about the use of slagcement;

1. Ground blast furnace slag could replace up to 50% of the portland cement in consolidated rockfill without loss of its compressive strength provided that the fill was allowed to cure for three months or more.

2. The mix of portland cement and slag with sand slurry produced an acceptable fill but required longer curing time to obtain the desired strength.

3. The use of slag cement presented no operational problems.

In Kidd Creek No.1 Mine, where a minimum three-month curing period was available before adjacent pillar recovery, up to 60% of the Portland cement was replaced by an equivalent weight of ground slag with satisfactory results. Due to a shorter curing period available in the No. 2 Mine, normally three weeks, only 33% replacement of Portland cement by ground slag was allowable. From 1982 to the end of 1984, a total of 70,000 tonnes of ground slag was used in backfill at Kidd Creek Mines.

5.6.2 Flyash

Flyash is a by-product of the combustion of pulverized coal in thermal power plants. It is removed by mechanical collectors or electrostatic precipitators as a fine particulate residue from the combustion gases. As a waste product that does not require grinding, flyash is available at a considerable price discount compared to portland cement and even ground blast furnace slag.

Commercial flyash can be classified into two types, type F and type C. Type F flyash, produced from bituminous coals, has a low lime content and possesses little cementitious value by itself; however in presence of Portland cement, it slowly combines with the calcium hydroxide released during the cement hydration process to form new cementing compounds. Type C flyash, produced from sub-bituminous or lignite coals, contains a higher lime content than type F flyash, and possesses some cementitious qualities of its own. It will chemically react with water, even if there is a deficiency of portland cement. Hence, the setting time is faster than for type F flyash.

Kidd Creek began test work in 1973 on a partial Portland cement replacement by type F flyash in consolidated fill. The results indicated that flyash had the potential to be considered as a partial portland cement replacement. However, the requirement of longer curing time periods and the inconsistent chemical composition of flyash impeded its use. Also, at that time, cement prices were low, and there was not a significant cost advantage to justify the use of flyash in the backfill. In 1983, however, the steadily increasing costs of portland cement and ground blast furnace slag prompted the search for a lower cost cementing agent.

Four flyash samples from different sources were evaluated. Of the four samples, two were Type C ash, and two were Type F ash (Yu and Counter, 1988). Three series of tests were conducted. The first test series evaluated the cementitious properties of one Type C flyash sample and two Type F flyash samples, in comparison with ground blast furnace slag. The second test series confirmed the strength development of Type C flyash. A total of over 200 test cylinders, 15 cm in diameter by 30 cm high, were cast for uniaxial compression tests after 28, 56 and 85 days of curing. The third test series compared the two Type C flyash samples. In addition, 60 cm cubes were cast for relative dynamic strength evaluation.

The results of the tests were;

1. Type C flyash exhibited higher strength than the type F flyash at every mix proportion and curing period.

2. The compressive strength of the 30% type C flyash was the same as that of the 30% slag mix at 28 days curing and became higher afterwards, surpassing the portland control mix after 43 days of curing.

3. The 60% type C flyash mix had a lower 28 day strength but got steadily stronger and exceeded both the 60% slag mix and the portland control mix after 73 days of curing.

4. There was no indication of any advantage in combining the flyash with the slag.

5. Development of strength for both type C flyashes trialed was very consistent.

To fully evaluate the performance of flyash as a partial portland cement replacement a full scale stope test was carried out in 1984 with the following findings;

1. Flyash could be pneumatically loaded into the silos from tanker trucks in the same manner as

portland or slag cement, with no difficulty.

2. The behavior of the flyash was similar to that of slag during the weighing, mixing, and clean up phases in the cement slurry mixing plant.

3. No accumulation of undue scale build up was noted with using flyash.

4. Type C flyash exhibited comparable cementitious properties as ground blast furnace slag.

5. The emanation of radon gas, in terms of Working Levels, from all tested flyash mixes was within Ontario Ministry of Labour guidelines.

The results indicated that flyash could replace up to 60% of the portland cement as a binder and the rockfill strength would not suffer, indeed it was often superior. The strength development was a little slower so the rate of portland cement replacement would have to be judicially controlled.

As a result of research work Kidd Creek Mines have replaced up to 60% of the portland cement used as rockfill binder with type C flyash. A flyash and portland cement blend is used as their current binder. With this blend the binder costs are lower than both portland cement and blast furnace slag alternatives.

5.6.3 Kidd Creek Mines Copper Slag Research

The challenge in backfill research is to match or exceed the strength achieved with current materials by using less expensive ones. As part of this project Kidd Creek Mines have conducted extensive test work into the potential of local Kidd Creek Mines copper slag as a binder alternative. The test work has indicated that the slag can be made into an acceptable binder, but it is not cost competitive at the present time with the flyash portland cement blended binder.

The economics of copper slag usage as a binder are directly related to the cost of grinding the slag to a size fine enough to make it sufficiently reactive. Fineness of a cementitious material is generally quoted in terms of Blaine size, a measure of the available surface area per unit weight of material, (units of m^2/g). A considerable volume of test work has been done on the costs involved in reducing the Kidd Creek Mines copper slag to a suitable Blaine size, in the order of 3000. Test work described in section indicated that the reactivity did not improve significantly at grinds finer than 3000 Blaine.

The power consumption needed to grind the Kidd Creek copper slag to 3000 Blaine size has been estimated to be 80 KWh/tonne. This work was conducted as part of this project. CANMET and Kidd Creek Mines are still working together on methods of decreasing the cost of this grinding (McIvor et. al. 1990). Most emphasis has been placed upon dry grinding similar to that practiced in the portland cement industry. Some operations employing slag cements in mine backfill are using much cheaper wet grinding technology (ball mills) with success (O'Kiepe Mine South Africa). The wet ground slag is mixed immediately with the backfill for placement underground.

An extensive program of testing conducted under the auspices of this project indicated that copper slag blended with portland cement made an acceptable binder, that it was best to have a portland cement to slag ratio of 0.5, and that the addition of anhydrite significantly improved the performance of such a blend. A principal conclusion from this project is that Kidd Creek copper slag can be used to make an acceptable backfill binder. The process of testing and the performance of various additives in these tests has implications for other ferrous and non ferrous slags either currently being used or tested for use as backfill binders. A discussion of this work and the results are presented in Chapter 8.

5.6.4 Binder Alternatives Test Work

As part of this research project a comprehensive testing program was carried out to evaluate the cementitious properties of different binders with the addition of commercially available chemicals. The main emphasis in this testing program was to evaluate the cementitious properties of copper slag as discussed above. Increasing the early strengths of the flyash with portland and slag with portland blended binders was also extensively studied. The results of this research is presented in Chapter 8 on Binder Alternatives.

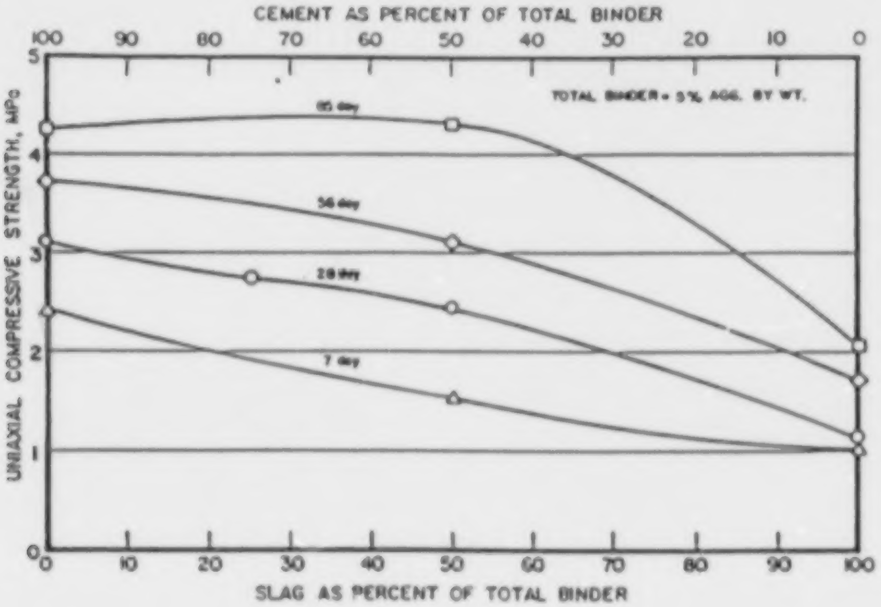


Figure 5.9, Ferrous blast furnace slag as a rockfill binder at KCM, UCS (MPa) versus addition rate.

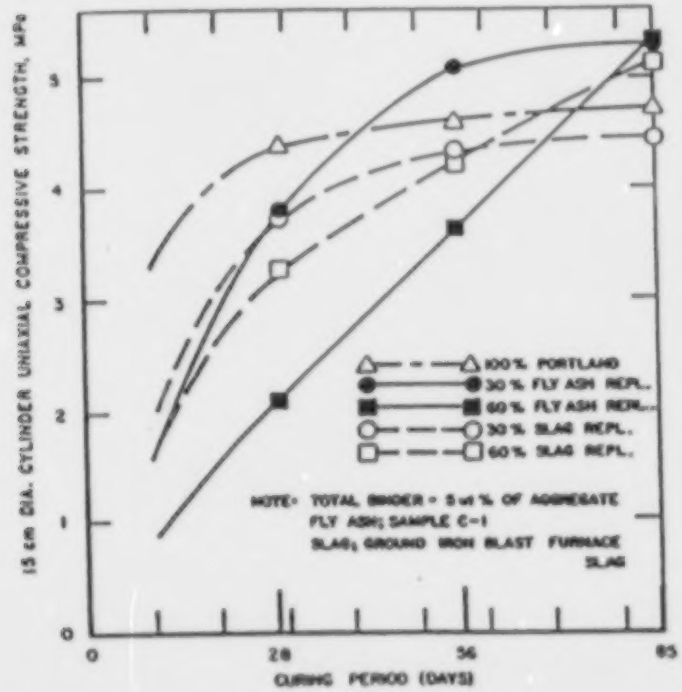


Figure 5.10 Results of rockfill binder alternative testing at KCM.

5.7 QUALITY CONTROL

Consolidated rockfill attains optimum strength when properly blended aggregates which are well coated with binder material are placed in the stope. The aggregate must be of the appropriate quality and sized according to design. In addition the binder must be of the correct blend and supplied in the correct proportion.

The quality control of materials used in the rockfill system can be considered in three stages;

1. At the backfill plant ,
2. During transportation and,
3. During placement underground.

5.7.1 At the Fill Plant

5.7.1.1 Aggregate and binder

A rockfill preparation plant, the same as a concrete plant, should provide properly sized aggregate by precise weight or volume and the correct quantity of binder for this aggregate.

When consolidated rockfill is used as the filling material, excessive fines created by the attrition of aggregates in a long raise will reduce the fill strength. Attrition can cause a significant difference between the average size of aggregate that is sent underground from the fill plant and that actually placed in the stope. The effect of attrition for different levels of the mine should be estimated and proper size aggregates for each level should be produced on surface.

A properly sized aggregate will produce an optimum fill density underground. To compensate for the attrition effect, the initial average aggregate size should be larger than the design placement size. The common method of doing this is to reduce the proportion of fine material in the aggregate by screening in the surface plant. Correct blending of coarse, middle sized and fine material in the backfill aggregate will translate into higher placed density of the fill, less voids and lower cement consumption.

The cementing agents should not be stored in bins or hoppers for long periods of time in moist environmental conditions due to moisture and time consolidation effects.

5.7.1.2 Quantity of water

The first step in maintaining a good quality backfill is minimizing the water content of the mixes by having the highest possible pulp density binder slurry. Strength development in concrete is significantly affected by the water to binder ratio; lower the ratio the higher the strength achieved so long as there is still sufficient water for complete hydration. In rockfill the same holds true. However rockfill placement is hampered by the usually less than ideal mixing arrangements which prompt the use of excess water to ensure the binder slurry is sufficiently fluid to coat all the rock particles. There is small tolerance on how much water should be used; too much and the water binder ratio is excessive or the binder tends to be washed off the rock, too little and the binder slurry will not flow well enough to coat all the particles.

Adding separate (apart from binder slurry) water to the aggregate should be avoided. Excess water in the slurry or aggregate will wash the cement paste and the cement coating off the aggregate, and flush it toward the lowest zone of the stope. This causes dilution of cement content in the fill pile resulting in a less homogeneous fill mass. A slight change of moisture content in the aggregate may affect the fill quality significantly. To prepare a normal batch of consolidated rockfill at Kidd Creek requires only that amount of water equal to 3.9 % by weight of aggregate. An increase of 1 % moisture content in aggregate results in an excess of 25% mixing water. Close observations of fill piles during stope filling is therefore essential to the adjustment of the pulp density of cement slurry for a proper mixing of fill materials. Kidd Creek Mines have noticed that during the winter season poor coating and delayed initial curing were caused by frozen aggregate. To alleviate this situation, calcium chloride was mixed in the aggregate, at a rate of 2 % of the binder by weight, to lower the freezing point of the aggregate by 12 deg. Celsius, and to provide additional heat for curing.

5.7.1.3 Quality of water

An important factor in obtaining good quality fill is the quality of the water used. The same water quality criteria applied in the preparation of concrete should be used in the production of rockfill. As an example, some fill wall failures experienced at Kidd Creek Mines have been traced to the use of recycled underground waste water for mixing the backfill slurry. Upon close inspection of the failed backfill, it was observed that the slurry coating the aggregate was still "green" and had not properly cured. Several possible causes for the lack of proper curing were examined. After a comprehensive investigation, it was concluded that the the quality of the water used was the most feasible explanation for the low strength of the fill.

Laboratory test work conducted as part of this project confirmed the underground observations on the importance of water quality on rockfill strength. Cylinders containing a 60 % flyash / 40 % portland cement binder blend were cast using underground recycled water. A 50 % loss of compressive strength compared to control cylinders cast with fresh potable water was recorded, (Henning, 1988).

The tests also indicated that the cylinders cast using underground recycled water were much less cohesive and bled more than cylinders cast with potable water. These results suggest that the hydration of portland cement and flyash was hindered by the recycled water, with the result that water required for hydration was released as bleed water. Hydration of portland cement with the resulting release of calcium hydroxide is required to activate the cementitious properties of the flyash. The effect on strength development of a reduction in the degree of the portland cement hydration will be more pronounced in a 60 % flyash / 40 % portland cement blend than in a 100% portland cement blend. Contaminants found in the underground recycled water which might have caused the reduction of the backfill strength included;

1. high concentrations of dissolved solids,

2. buildup of oil and grease,

3. buildup of water treatment chemicals.

5.7.1.4 Instrumentation

Most rockfill plants have a combination of different electronic and mechanical hardware to monitor and produce the required backfill materials. All the equipments such as pumps, valves, flow meters, density meters and binder measuring devices should closely meet with their designed requirements. The emphasis should be placed on weighing the right amounts of binder, and on obtaining the proper pulp densities for the slurries used underground. (Lausch, 1989)

To be able to produce a good quality fill mixture on surface all the devices for monitoring material consumption, rates of transfer, storage quantities, etc., should be accurately maintained, and the calibration checks on the devices should be carried out at a specified interval. If regular checks are not carried out deviation from the designed quantity can easily occur without being noticed. For consolidated rockfill, any overdosed aggregate or insufficient amount of binder can cause a shortage of binder slurry available to coat each solid particle, resulting in a weak fill structure. An overdosed binder or lower proportion of aggregate will produce a higher strength fill but at high cost.

The last step in checking the material quality in the fill plant is to conduct regular sampling of the final products. This should be done to verify and detect any long term deviation in quantity and quality of the fill materials prepared by the fill plant.

5.7.2 In Transport

The main quality control method used for conveyor transportation is close observation of the sizing of aggregate on the belt by the underground operators . This is used to identify when excessive fines are being placed in the stope. It also enables the operator to act rapidly in response to process changes, for example having an extra batch of slurry if the material on the belt is too fine. Weightometers and speedometers on the conveyor belts should be regularly calibrated.

If the rockfill is hauled by truck or scooptram, the amount of fines in the aggregate should be closely monitored. A material with a high fines content may be subject to differential settling in the truck tray or scoop bucket.

Another aspect to consider with truck transportation is to make sure that the operators load the truck with the same amount of aggregate for each slurry batch. Since the only measuring quantity when filling the truck is by the volume of the box, obtaining a consistent aggregate load is essential.

5.7.3 In Placement

The major quality control problem when filling with consolidated rockfill is the segregation phenomenon which occurs during placement, as explained earlier. The main factors which affect the extent of the segregation are stope geometry, aggregate size, filling system (conveyor, truck), and the orientation and dimensions of the fill raises.

The choice of stope geometry could be such as to minimize the segregation phenomenon by preventing the development of steep fill cones. However in the majority of cases alteration of the shape of the stope just for the purposes of filling is difficult to justify. The general practice in filling an open stope involves a single fill pour point. In a larger stope a second pour point should be available to minimize the rolling distance of the coarser aggregates toward the walls of the stope.

The first step in maintaining a good quality backfill is minimizing the water content of the mixes by having the highest possible pulp density binder and sand slurries.

When a stope is being filled with consolidated rockfill there is little concern about drainage problems due to the low overall water content of the final fill product, 5 % water content in cement slurry and 1-3% moisture in the aggregate. Drainage facilities might be needed, however, when sandfill is added to increase the blast vibration resistance or to give tight filling.

Close observations of fill piles during filling are necessary for quality control. Daily inspection should be carried out to monitor the fill quality. As there is generally very limited access to evaluate the progressive filling of any stope, all possible accesses should be utilized, e.g., by having windows in all bulkheads above the floor of the stope. The daily records of the tonnages of fill materials placed underground should be reviewed and corrections made if necessary.

To minimize blast damage to adjacent fill material, care must be taken to avoid blasting in the fill; by leaving a thin skin of ore when mining an adjacent stope, by preventing production holes intersecting the fill, and by stemming the holes should an intersection occur.

5.8 STRUCTURAL ROCKFILL DESIGN

Factors to be considered when structurally designing backfill include;

1. Orientation of the backfill raise.
2. Extent of segregation.
3. Degree of success in mixing the aggregate and slurry.
4. The extent of impact damage due to the free fall height.
5. The size of the aggregate entering the stope.

Once the backfill material has left the belt or been dumped from the truck into the stope, limited control or remedies are possible. Close observation of the state of the fill during placement may indicate the need for changes such as pulp density of the slurry, amount of fines in the aggregate and addition of extra binder slurry to lower the extent of segregation.

5.8.1 Using the Segregation Phenomenon

Structural differences developed when filling a stope with consolidated rockfill are mainly due to the segregation phenomenon. Understanding the mechanisms of segregation allows the designer to control the structure of rockfill to a moderate degree. This generally concerns large stopes filled by pouring from above through raises.

Segregation of consolidated rockfill is unavoidable but it can be minimized and used to advantage if fill operations are well planned and closely monitored. As mentioned earlier, fill in a raise attains a specific horizontal and vertical velocities which determine the trajectory of the fill in the stope.

5.8.2 Fill that Impacts the Stope Wall

When fill material leaves the fill raise, the trajectory of the fill may be such that it will collide with the wall of the stope above the floor. This collision will cause two things to occur;

1. A portion of the slurry which was coating the aggregate will be knocked off and run down the wall,

2. Due to the greater momentum of the larger particles, they will rebound further in a horizontal direction compared to finer particles.

Because of these two occurrences the area below the impact zone and on the same side of the stope which the collision occurred will tend to be the strongest side of the stope. Points A & B on figure 5.11a

If we consider a point above the level of impact zone, point B on figure 5.11a, four things will occur;

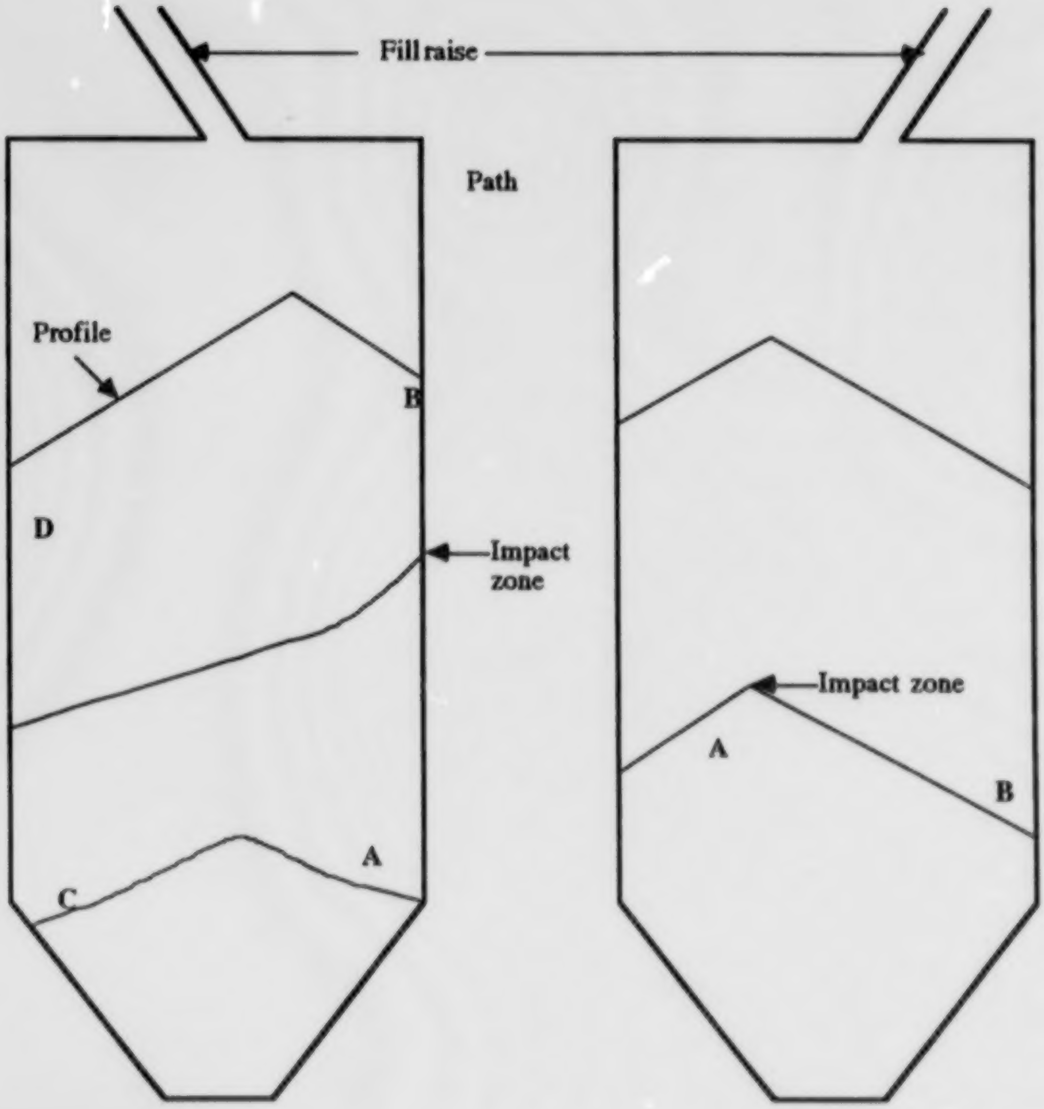
1. Excess slurry or water will fall short of the peak of the fill and have a tendency to flow down the slope of the fill in the direction of the fill raise. While the slurry is flowing down the slope it will carry some of the fines with it.

2. Due to having higher momentum the larger particles will rebound further horizontally than the finer particles after impact. This will lead to an accumulation of coarse particles on the wall of the stope.

3. Upon impact a portion of the coarse particles will bury themselves rather than rebound. Some of the coarse material will remain in the backfill cone area.

4. Due to the effects of friction and cohesion in the raise, as well as air resistance in the stope, the finer particles neither leave the raise with the same velocity nor have the same acceleration in the stope as the coarser particles. This accounts for the fines not traveling the same horizontal distance in the stope and thus magnify the effect of an increased percentage of coarse material on the side of the cone towards the wall which the backfill previously collided with.

Point C in figure 5.11a represents a location where an accumulation of coarse particles is expected. The appearances of the backfill at this point can be like either coarse concrete, due to excess slurry washing a portion of the fines down from the fill peak towards this wall, or unconsolidated conglomerate pile with no excess slurry to fill the voids.


Fill raise
Path
Profile
B
D
Impact zone
A
C
Impact zone
A
B

a) Fill impacts stope wall.

b) Fill does not impact stope wall

Figure 5.11, Structural profile of rockfill in large stopes filled from above.

The rockfill placed under the configuration shown in figure 5.11a would be expected to show the following structural characteristics;

1. The wall on which a collision occurred and below the impact zone, point A, is like concrete and has the highest in-situ strength.
2. The wall on which a collision occurred and above the impact zone is partially segregated, point B.
3. The wall opposite the one on which a collision occurred and below the profile of the fill at the impact zone is like coarse concrete or conglomerate, point C.
4. The wall opposite the one on which a collision occurred and above the profile of the fill at the impact zone is highly segregated and has the lowest in-situ strength, point D.

The most critical portion of a backfilled stope is the wall(s) which will be exposed for future pillar recovery. The fill raises(s) should be placed such that the fill will collide as high as possible on the wall(s) which are required to stand. This will utilize the segregation phenomenon so that the wall below the impact zone is the strongest part of the filled stope.

If the filled stope is to be exposed at more than one wall it is important to have multiple raises. By having a multi raise system the fill could have a more uniform distribution and a more horizontal profile. This would eliminate the existence of a rill surface which is a plane of weakness.

5.8.3 Fill that Doesn't Impact the Stope Wall

In the case where the fill material does not collide with the stope walls, fine backfill aggregates accumulate at the fill cone and have a high cement content, producing a high quality fill, (point A on figure 5.11b). Because of the higher momentum of the coarser particles, they segregate towards the boundary of the stope resulting in a low cement content, poor quality backfill (point B, figure 5.11b). Observations underground confirm this. Since higher strengths are obtained under the impact point, dump points should be positioned around the perimeter of the stope.

The fill quality and extent of segregation at the boundaries of a stope are related to the rolling distance over which the coarse particles have to travel before colliding with the walls. Lack of cement and fine fractions at the boundaries will have an adverse effect on the fill quality. Some portion of the coarse particles will, however, bury themselves in the material at the impact zone, due to the high impact velocity, especially when using high speed conveyor belts for placement. If the orientation of the fill raise(s) does not allow the fill material to collide with the stope wall, the dip of the fill raise(s) should position the cone area as close as possible to stope walls that are to be exposed in future pillar recovery. This will translate into having more cement content, higher fine fractions and less segregated material close to the perimeter of the stope .

As noted earlier, if all stope walls are to be exposed for future pillar recoveries, then multiple vertical raises yield the best possible distribution of the fill materials. The vertical raises produce a more horizontal fill profile and any excess cement slurry will accumulate at the stope boundaries. If vertical raises are not possible, then the number of inclined fill raises should be increased to produce a competent fill at the critical stope walls.

5.8.4 Angle of Repose

On occasion, the angle of repose of the fill during placement has to be adjusted so that the resulting slope will allow ore to run as an adjacent pillar is blasted and mucked. Repose angle can also be adjusted so that tight filling is achieved in enclosed areas. The typical angle of repose for dry aggregate is 38 to 40 degrees. For consolidated rockfill 36 to 38 degrees is a typical angle, but this varies with the pulp density and cement content of the binder slurry. For consolidated sand rockfill the angle of repose is between 18 to 25 degrees, and for cemented sand fill or sandfill alone it is 10 degrees. With the right design and the availability of hydraulic fill the profile at the top of a stope can be reasonably tight filled.

The main concern when planning a fill raise should not be the final profile of the fill but the backfill quality. The final fill profile can be altered, but dilution caused by the poor backfill quality may not be remedied.

5.8.5 Stope Geometry

The stope geometry plays an important role in controlling the segregation effect. The shape of the stope and the orientation of the fill raises dictate the rolling distance of the material after the impact with the fill cone. If the distance is kept small, such as in a small square shape stope with centrally located fill raise, then the coarse particles have a chance to rebound back to the vicinity of the impact area after colliding with the stope wall. This can result in a more uniform fill distribution and a more horizontal fill face in the stope. Should one or two walls be required to stand, the fill raise(s) should be placed such that the fill will collide as high as possible on the wall(s) required to stand. This will utilize the phenomenon that the wall below the impact zone is the strongest wall.

If the rolling distance is increased due to an enlarged stope geometry or a rectangular shape stope, then the material has no chance of traveling the same distance after colliding with the wall. This will lead to the formation of low density weak fill at the wall of the stope.To overcome this effect, a multiple fill raise system is recommended. When four stope walls are required to stand, the backfill raises should be vertical and centrally located spaced at a distance of no more than the stope width. If vertical fill raises are not possible, the number of the fill raises should be doubled to ensure that high strength fill can be placed against all walls.

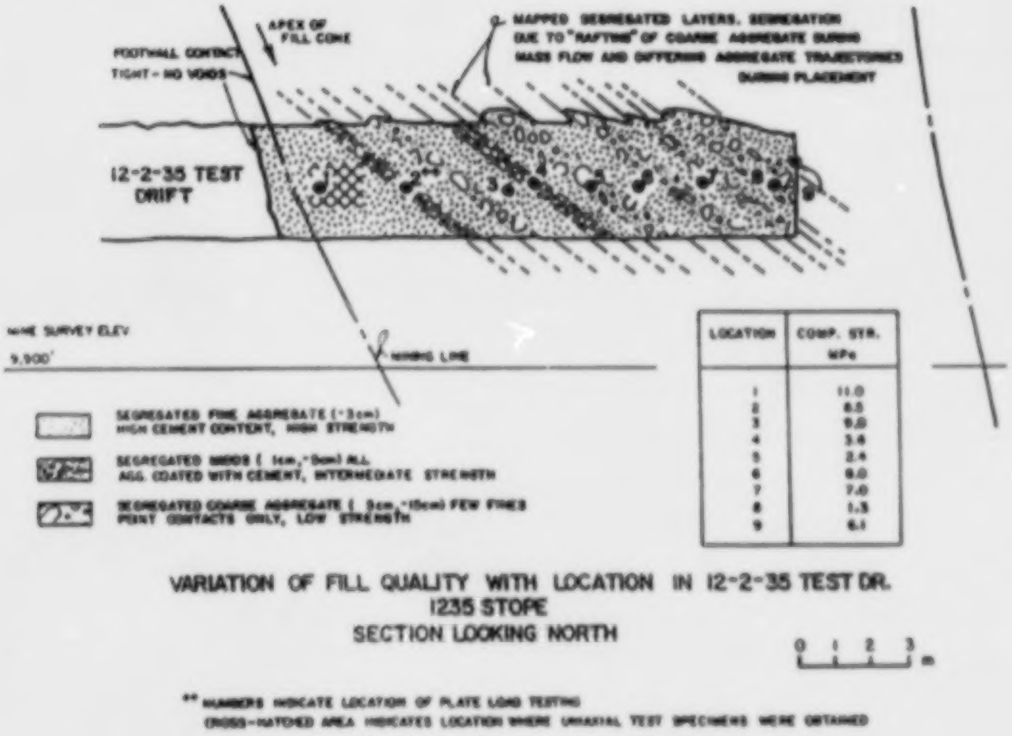
If possible the fill free-fall height should be kept as small as possible, by having pour points at different levels. This will minimize the amount of energy possessed by the aggregate at the time of colliding with the fill cone, which causes breakage of the already cementing bonds and the aggregate particles themselves. The energy of falling causes an increase in the already high fine particle fraction at the impact area and possible formation of impermeable layers which limit the percolation of slurry. The fines also coat the coarser particles preventing proper bounding between them. (Lausch, 1989)

5.8.6 Investigation of In Situ Structure at Kidd Creek Mines

To simulate the actual filling operation, several large scale dump tests were carried out at Kidd Creek Mines. The results revealed an adequate strength of over 6 MPa, with apeak strength of 10.3 MPa in specimens cored from 838 stope. However, it was found from chemical analysis that the cores contained a high cement content, 7.3% as compared to the average 5%. This indicated that cement concentrations could occur in some areas of large fill piles, resulting in undesirable deficiencies of cement in other areas.

At Kidd Creek Mines, a test drift 14 m long was driven in a stope which had been filled one year earlier with consolidated rockfill. The drift was excavated to evaluate the support requirements of excavations in the fill material. The drift also enabled various in-situ strength tests to be performed. Observation of the drift gave insight into the distribution of consolidated rockfill in a stope. It was observed that the fill exhibited alternating layers of coarse and fine aggregate resulting from slumping and uneven aggregate distribution during placement. From a series of plate-load test and laboratory strength tests, on samples obtained in the drift, the in-situ strength of the fill material was determined, as shown in figure 5.12. It was observed that the strongest fill, with an average strength of 11 MPa, was obtained within the dump cone area. The strength decreased as the aggregate in the layer got coarser. From the distribution characteristics of the fill and the relationship between the cement content, it is clear that a zone of a high cement content tends to occur near the dump cone area, leaving a lean cement rockfill at the perimeter of the fill cone, (Yu and Counter, 1983).

To minimize the effect of segregation without increasing the number of fill dump points, the addition of sand to act as a void filler was tested. The sand had 90 to 95% by-weight passing a No. 18 mesh (1.0 mm) screen, and an average percolation rate of 15 cm/hr. This sand when mixed with cement slurry, became an ideal material to fill the voids in segregated aggregate or waste.


APEX OF FILL CONE
FOOTWALL CONTACT TIGHT - NO VOIDS
MAPPED SEGREGATED LAYERS, SEGREGATION DUE TO "RAFTING" OF COARSE AGGREGATE DURING MASS FLOW AND DIFFERING AGGREGATE TRAJECTORIES DURING PLACEMENT
12-2-35 TEST DRIFT
1
2
3
MINE SURVEY ELEV
9,900'
MINING LINE
SEGREGATED FINE AGGREGATE (-3cm) HIGH CEMENT CONTENT, HIGH STRENGTH
SEGREGATED [illegible] (1cm,-5cm) ALL AGG. COATED WITH CEMENT, INTERMEDIATE STRENGTH
SEGREGATED COARSE AGGREGATE (3cm,-15cm) FEW FINES POINT CONTACTS ONLY, LOW STRENGTH
VARIATION OF FILL QUALITY WITH LOCATION IN 12-2-35 TEST DR.
1235 STOPE
SECTION LOOKING NORTH
0 1 2 3 m
** NUMBERS INDICATE LOCATION OF PLATE LOAD TESTING
CROSS-HATCHED AREA INDICATES LOCATION WHERE UNIAXIAL TEST SPECIMENS WERE OBTAINED

LOCATION	COMP. STR. MPa
1	11.0
2	8.5
3	9.0
4	3.6
5	2.4
6	8.0
7	7.0
8	1.3
9	6.1

Figure 5.12,Variation of rockfill quality with location in 12-2-35 test drive, 1235 stope. Section looking North. (After Yu and Counter 1983)

5.9 ROCKFILL ECONOMICS

Traditionally rockfill was utilized in stopes to provide passive support by decreasing the ground movement around the filled stope. However, with the addition of binding materials, higher pillar recoveries, lower mining costs, decreased ore dilution and improved ground control were possible.

The major portion of the filling cost for a consolidated rockfill is the cost of the binder. Other costs are; labor costs for inspection, maintenance, stope preparation and transportation, and equipment capital costs such for pumps, mixers, instrumentation, storage facilities, pipelines and conveyor systems. In addition a certain amount of mine development costs must be apportioned to the backfilling operation for accesses and raises etc.

The best chance of saving costs in any consolidated fill system is to minimize the amount of the binding material required to produce the desired fill strength. The amount of binder material used for each stope may differ, and is related to the following;

1. stope and pillar grade and tonnage,
2. stope and pillar mining costs,
3. relationship between consolidated fill quality and % pillar recovery and
4. the consequential costs due to such things as production delays, minor or major fill failures, (Thomas et al., 1979).

Generally, for the recovery of a high grade pillar, every attempt should be made to minimize ore dilution and production delays caused by fill failure by placing a higher than average cement content in the fill against which the pillar is to be mined. For a low grade pillar recovery where mining costs are important factors, the cement content of the fill is kept to a minimum.

Methods which yield a lower overall cement cost for a given fill strength are;

1. Increased mining cycle time through proper planning, which gives the binder material a longer curing time. The consolidated fill materials get stronger with time, although, with a decreasing rate. This would allow a lower overall cement content for the required fill strength. For example, consolidated rockfill using a blended binder of flyash and portland cement at Kidd Creek Mines has 20 to 30% higher strength when the curing time has increased by a month, from 28 to 56 days curing times, (estimated from laboratory results). The rate of compressive strength increase depends mainly on the types of the cementing agents used.

2. Because of the lower costs of different local and/or commercially available pozzolanic materials, a portion of portland cement could be replaced with these materials without sacrificing the strength and quality of the fill. In recent years pozzolanic materials such as flyash and slag have successfully replaced a portion of portland cement in different fill operations resulting in significant savings. For example, the binder cost at Kidd Creek Mines for placing a tonne of consolidated rockfill containing portland cement and flyash binders is about 15% less than that with portland cement only, and the overall filling cost is between $10 to $12 per tonne, (1990 Canadian dollars)

5.10 STOPE CAPPING TRIALS

A major difficulty in the placement of rockfill, especially in large open stopes is getting the fill tight to the stope backs. Where tight filling is critical it is common to top up the rockfill with a layer of hydraulic fill which posses relatively good tight filling capability. There is considerable expense involved in this; a complete additional fill system is required, bulkheads are required, and additional time is required to fill a stope. Hydraulic fill placement for tight filling is not particularly efficient. A good portion of the fill placed will be absorbed in to the voids in the rockfill, this helps improve the dynamic strength of the rockfill but doesn't help in tight filling the stope.

As part of this project a number of trials were conducted by Kidd Creek Mines into the potential for different cementitious materials to act as more efficient stope capping materials, or their ability to help hydraulic fill do so.

5.10.1 Tekpak Material for Stope Capping

With the present tight filling arrangements sand fill slurry will run down the stope through the voids in the rockfill material resulting in considerable excess consumption of sand fill. The possibility of using commercially available materials as a means of sealing the top of the rockfill to permit more efficient tight sand filling was studied. A seal on the top of the rockfill would prevent the percolation of sand fill through the voids.

Tekpak is a brand name monolithic packing material which was tested as a capping material. Chapter 6 discusses monolithic packing materials in detail. They are cementitious materials formed from two components that when mixed react rapidly to form a hard product. Monolithic packing is commonly used in underground soft rock mining where it is used to form artificial roof support.

For stope capping the operator is looking for a material which has a reasonable standing time so that it can be mixed and then pumped into the stope to be sealed. It should flow well over the surface of the rock because good access to the stope may not always be available and it should set hard quickly once it is in place. The set material must be strong so that it does not become a weak layer in the fill mass and should be reasonably impermeable. Monolithic packing products seem to fit these requirements well.

The underground trials indicated that the Tekpak material could not act as stope capping agents if high pressure spraying or long vertical falls are required for placement. A poor seal was obtained when placed in this manner. However the material showed good sealing characteristics when a slow rate of pour was used. This slow placement would be hard to achieve in practice. Tekpak set immediately after the blending of its two components, and was solid in a very short time. The rapid setting time of these materials could find other applications underground where pre-made forms contain the final mix, such as in bulkheads. Another trial conducted as part of this project investigated just such an application, and is described in Chapter 6. The rapid setting time and consequent rapid loss of pumpability proved to be a problem in this application of the material. The practical difficulties in being able to mix, blend and then pump the product to the stope in the short time before it lost pumpability do not encourage its use as a stope capping material.

Coal mines are able to employ monolithic packing materials at reasonable cost because the large volumes they are using makes for economies of scale. However for the smaller volumes being considered for stope capping applications the material is as costly as concrete at the site underground and for most jobs it would require more material due to its lower strength.

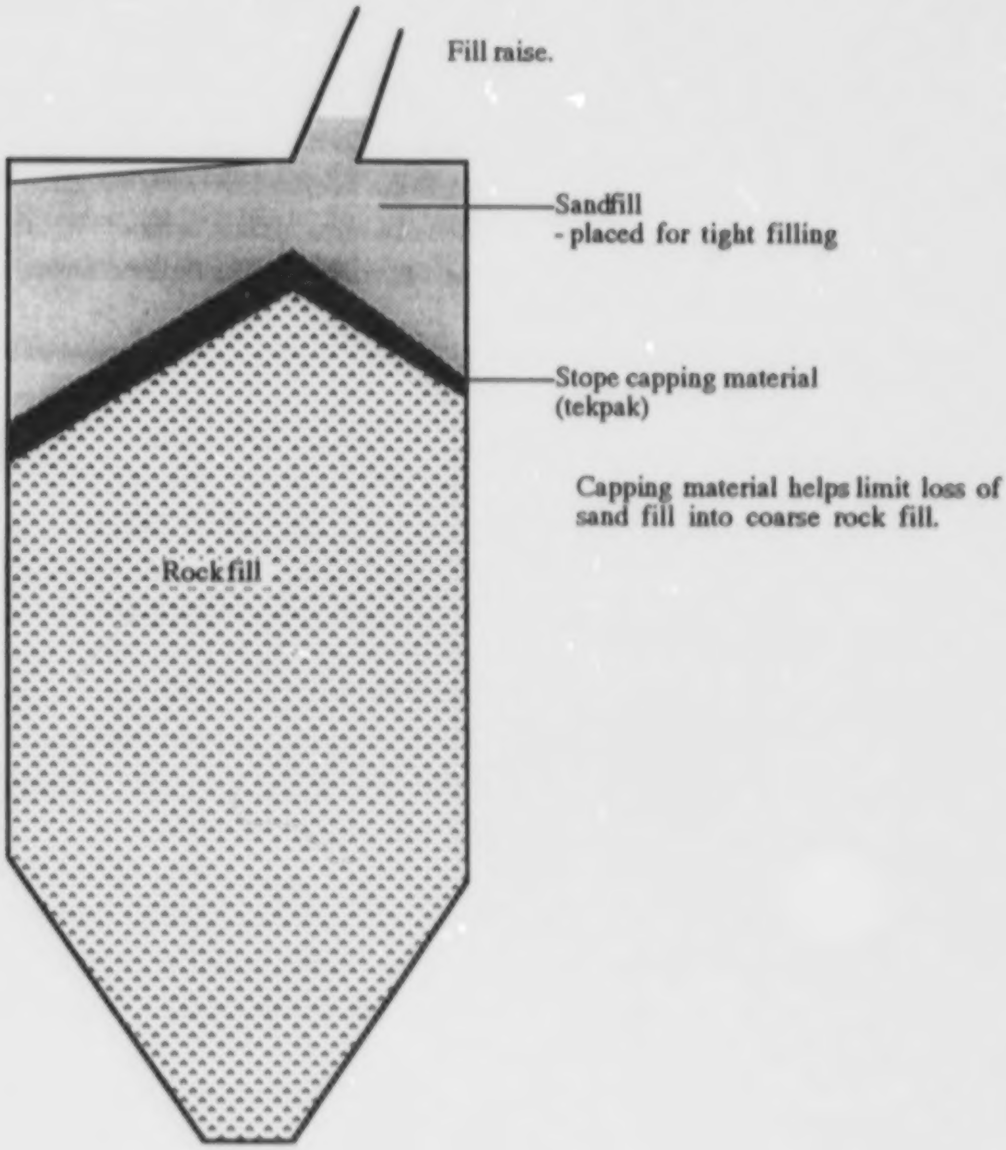

Fill raise.
Sandfill
- placed for tight filling
Stope capping material
(tekpak)
Capping material helps limit loss of sand fill into coarse rock fill.
Rock fill

Figure 5.13, Principle of stope capping trials with tekpak.

5.11 CHEMICAL ADDITIVES

5.11.1 Flocculants and Retarders

Retarders are used to delay curing rate and to reduce the effect of large pour volumes on the quality of the rockfill. At Kidd Creek Mines, retarders were used to minimize the impact damage and the extent of the broken bonds in previously placed cemented rockfill due to subsequent rockfilling.The retarder, however, has not been applied since the use of blended portland cement and slag or flyash was introduced. The blended binders exhibit a much slower curing rate compared to portland cement alone.

Flocculants are used to prevent local concentration of fine material, which produces an impervious layer.

5.11.2 Trials with Dispersants and Superplasticizers

Superplasticizers could be used to give a negative charge to the cement particles thus making them flow with a much reduced water content. A lower water binder ratio will yield a higher strength rockfill. As part of this project a series of trials was conducted to investigate the potential of this concept.

5.11.2.1 Hydrafill slurry dispersant

Hydrafill is product of W. R. Grace & Co. of Canada Ltd. It is described as a slurry dispersant and works by imparting a negative electrical charge to the binder particles which then repel each other causing uniform dispersion. The dispersion results in a more uniform slurry with better flow characteristics so that a denser pulp may be used, hence cutting down on the excess transport water required. A lower water binder ratio results which should lead to higher strength rockfill. In addition the better dispersed binder should have more effective surface area available for reaction and hence additional strength should develop.

These trials conducted by Kidd Creek Mines showed that Hydrafill did have a positive effect on strength. Hydrafill could permit a reduction in the binder consumption while maintaining fill strength. Results are presented below in Table 5.1 which shows the strength of cylinders of rockfill cast with a flyash and portland cement blend and Hydrafill and the strength improvement achieved over the control as a percentage.

	Strength Achieved MPa					
	14 Days		28 Days		56 Days	
60% F.A./40% P.C. Control pour	4.6		5.1		5.8	
Hydrafill added	9.1	+198%	9.9	+194%	11.6	+200%
Hydrafill - 5% less binder	7.1	+154%	9.3	+183%	10.1	+175%
Hydrafill - 7.5% less binder	6.6	+143%	8.1	+156%	10.1	+174%
Hydrafill - 10% less binder	6.1	+133%	7.8	+153%	9.4	+162%

Table 5.1, Strength of test cylinders with Hydrafill added.(% strength change shown)

The full test is described in appendix A.

A further series of tests were conducted with both Hydrafill and a superplasticizer, Authes S-613. The results were similarly encouraging as for the first test. The superplasticizer required approximately twice the chemical by weight to perform comparably to Hydrafill. This does not necessarily reflect the cost of addition.

5.12 CONCLUSIONS

The emphasis in the rockfill section of this report has been placed on the consolidated rockfill used in open-stoping mining method where pillars are mined alongside the placed backfill. To produce a stabilized rockfill mass the static and dynamic strengths of the backfill materials should be determined. The static strength of the fill product, e.g., compressive strength, is estimated by defining the height and width of the most likely fill exposure. It should be noted that the mechanical properties of consolidated rockfill mass are much different from the ones of the consolidated hydraulic fill system. Due to the segregation, attrition and impact damage effects in a rockfill system, large ranges of in-situ mechanical properties and densities exist. These factors should be considered when determining the strength requirements of the fill material and a suitable factor of safety has to be introduced.

Consolidated rockfill, like concrete, is strong in compression, yet weak in tension, especially in the outer zone of the fill pile or adjacent to the stope walls. Since it is believed that the blasting vibration resistance is closely related to the dynamic strength of the fill, the segregated zones in filled stopes should be reinforced to enhance the tensile strength. This could be done by addition of void fillers, such as sand slurry used at Kidd Creek Mines, in highly segregated zones. (Yu and Counter, 1983)

In the fill plant on surface, quality control measures should be taken to minimize the adverse effects of aggregate attrition and excess aggregate moisture content. Attrition would cause a great deviation between the original size of the aggregate on surface and the size of aggregate received underground. For mining operations using rockfill, the attrition effect for different levels of mine should be estimated. By estimating the attrition effect, crushing and blending could be utilized to produce a quality, graded product, suited to requirements at a certain depth. To control the attrition effect, the initial aggregate size could be larger than the required aggregate size for stopes at certain depth. However, a more common method of decreasing the adverse effect of attrition is to reduce the fine fraction by screening out the excess fines in the final mix underground. Adequate blending of coarse, midds and fine material in the backfill aggregate would translate to placing a higher density fill with less voids and, in turn, reducing cement requirements.

The moisture content of the stockpiled aggregate should also be closely monitored. A slight change of moisture content in the aggregate may affect the fill quality significantly. If the aggregate is sent underground wet or becomes wet on route, the result is that the fines will coat the coarser particles preventing the bonding of coarser aggregate and cement. Another important factor for obtaining good quality fill product is the use of quality water to prepare cement and/or sand slurries for consolidating aggregates.

All the devices in the surface fill plant for monitoring material consumption, rates of transfer and also storage quantities, should be accurately calibrated and the calibration checks should be carried out in short and regular intervals.

The main quality control measure with hydraulic transportation is maintaining proper pulp densities for cement and/or sand slurries. The water content in the slurries should be minimized to obtain the highest pulp densities possible. The quality control measure when using conveyor transportation method is to closely observe the sizing of the aggregates. This enables the underground operator to act rapidly if any process changes may be required, for example asking for extra slurry if the aggregate on the belt is too fine.

When filling a stope, segregation of consolidated rockfill is unavoidable but it can be minimized if the fill operation is well planned and closely monitored. For each individual stope following factors should be considered when structurally designing backfill : orientation of backfill raise(s), extent of segregation, degree of success in mixing the aggregate with slurry, the extent of impact damage due to the free fall height and the size of the aggregates entering the stope.

The most critical portion of a backfilled stope is the wall(s) which will be exposed for future pillar recovery. The fill raise(s) should be placed such that the fill will collide as high as possible on the wall(s) of which is required to stand. This will utilize the phenomena that the wall below the impact zone is the strongest wall. In stopes where more than one wall will be exposed for future pillar recoveries multiple raises should be used. By having multi raise system the fill could have a more uniform distribution and a more controllable fill profile. The filling of the stope should alternate between the raises. This practice will result in a more uniform distribution of cementitious material in the stope and also a much stronger fill at the boundaries of the stope which is desirable for future blasting.

The fill quality and segregation extent in boundaries of each stope are related to the rolling distance that the coarse particles have to travel before colliding with the walls. If the orientation of the fill raise (s) does not allow the fill material to collide with the stope wall, the dip of the fill raise(s) should position the cone area as close as possible to stope walls that are to be exposed in future pillar recovery.

As noted earlier, if all stope walls are to be exposed for future pillar recoveries, then multiple vertical raises yield the best possible distribution of the fill materials. The vertical raises would produce a more horizontal fill profile and any excess cement slurry will accumulate at the stope boundaries. If vertical raises are not possible due to the available access, then the number of inclined fill raises should be increased to produce a competent fill at the required stope walls.

The stope geometry plays an important role in decreasing segregation effect. The shape of the stope and the orientation of the fill raises dictate the rolling distance of the material after the impact with the fill cone. If the distance is kept small, such as in a square shape stope with centrally located fill raise, then the coarse particles have a chance to rebound back to the vicinity of the impact area after colliding with the stope wall. This could result in a more uniform fill distribution and a more horizontal fill face in the stope.

If possible the fill free-fall height should be kept as small as possible, by utilizing pour points at different levels. This would minimize the amount of energy of the aggregates at the time of colliding with the fill cone which causes further breakage of the material .

CHAPTER 6

SPECIAL APPLICATIONS AND INNOVATIONS

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6.1 INTRODUCTION

Conventional fill systems are characterized by compromises between in situ performance, cost, and the requirements of distribution and placement. A number of innovative ideas for improving backfill systems have been proposed. Some of the ideas have remained in the conceptual stage while others have evolved into operating systems.

As part of the broad scope of this project a number of special and innovative backfill materials and systems have been reviewed and investigated. A number of trials of different fill materials and additives were conducted.

1. Literature review of paste and dense hydraulic fill.
2. Literature review of pneumatic fill.
3. Literature review of frozen backfill.
4. Literature review of cementitious foam.
5. Centrifugal modelling to determine the potential for reinforcement of hydraulic fill with mesh and fibre elements.
6. Centrifugal modelling to determine the potential for layered, sloped wall and dense wall fill designs. Also a follow up stope trial of the layered fill methods.
7. Trials with monolithic packing materials for use in stope capping and fill containment barricade construction.
8. Trials with chemical additives to improve the early strength and water retaining properties of hydraulic fill.

A major part of this project was the subcontract let to Kingston Georesearch Limited to conduct centrifugal modelling studies on the performance of various backfill innovations. This work has helped to establish centrifugal modelling as a realistic design tool. A condensed version of the modelling results are presented here, the full reports are available in the appendices.

Probably the most significant innovation for the Ontario hard rock mining industry reviewed here are the concepts of high density hydraulic and paste fill. By reducing the volume of water used in hydraulic backfill a stiff, higher strength fill can be placed. The preparation and transport of these low water fills however presents some practical problems which are the subject of continuing research. There are a few mines operating these low water fill systems, but the technology is still very new. This chapter presents a short discussion of the technology involved.

A short review of pneumatic and frozen backfilling is presented. No research work was conducted on these systems, but they have potential for special applications and were investigated as part of the literature search for the project.

6.2 HIGH DENSITY HYDRAULIC AND PASTE FILL

6.2.1 Definitions

It is possible to produce a high strength, low binder content (i.e. lower cost) and rapid setting, hydraulically transported backfill with minimum drainage requirements. These paste like fills are produced by dewatering to a high solids concentration prior to placement. Various classes of hydraulic backfill with high solids concentration are defined according to their physical characteristics and flow regimes. The following definitions have been adopted for this report and are compared to conventional hydraulic fill.

1. **Conventional Hydraulic Fill**
    - has a pulp density much lower than the settled density , usually limited to 70% for most operations,
    - pipeline transport with slurry flow in the turbulent regime,
    - excess delivery water must be drained so that permeability is a critical design parameter,
    - may have a size distribution which is non optimum from a consolidated strength perspective,
    - average consolidated strength,
    - small angle of repose when placed.

2. **High Density Hydraulic Fill (Dense Slurry Fill)**
    - density of slurry just below that of the settled density,
    - pipeline transport in the turbulent flow regime,

- sufficiently low viscosity that they can be pumped with centrifugal pumps or can be gravity fed,
- have some excess transport water to drain but permeability is not a critical design parameter,
- may have a particle distribution which is optimum for strength development in consolidated fill, (higher fines content allowed),
- forms an average to high strength fill from fine material.

3. **Paste Fill**
   - have a very high pulp density, greater than the natural settled density of the material,
   - pipeline transport with flow in the laminar regime, (like toothpaste),
   - high viscosity and hence high resistance to flow,
   - when pumping required needs positive displacement pumps,
   - has essentially no excess water to drain when placed,
   - does not require containment barricades,
   - has an angle of repose around 65°,
   - may have a particle distribution which is optimum for strength development in consolidated fill.
   - forms a high strength, stiff fill from fine material.

4. **Dense Paste Fill**
   - paste fill with additional coarse aggregate,
   - aggregate makes a very high density fill,
   - forms a very high strength fill.

The distinction between a high density hydraulic fill and a paste type fill is often a grey area. Probably the most descriptive distinction between the two is paste fill will have a measurable slump (as in concrete technology). A high density hydraulic fill is too fluid to develop any slump.

6.2.1.1 Stiff fill

A stiff fill is considered to be one that has high modulus of elasticity compared to the values obtainable with conventional hydraulic fill. Hence one would consider high density hydraulic, paste and dense paste fill to be stiff fill. The fill stiffness improves in the same order. A stiff fill will always contain some binder to give it internal cohesion and prevent the possibility of liquefaction.

6.2.2 The Incentive for Development of Low Water Content Fills

Conventional hydraulic fill is characterized by a number of design compromises which result in it being less than an ideal fill. However the advantages of of pipeline transport and its consumption of an otherwise troublesome waste material make it still the most popular backfill type. The excess water required for transport of conventional hydraulic fill is the root of most of the contradictions in the engineering of a hydraulic fill. These problems were discussed in chapter 4. Low water content hydraulic fills have been developed to retain the advantages of pipeline transport and waste disposal/but provide a engineered fill at low cost and with high strength.

1. Low water content results in a good water to binder ratio and hence optimum use of the binder.

2. Minimum in place porosity with no excess water. Optimum strength due to higher internal friction angle. The relationship between pulp density and in place porosity is well recognized in conventional hydraulic filling.

3. Minimum drainage water means that permeability is no longer an over riding design criteria. The fill material may be sized for optimum strength development. Fine particles may be left in the fill reducing a surface waste disposal problem.

A number of lesser advantages from low water content hydraulic fills are recognized.

1. Less drainage water means less load on mine dewatering systems, limited requirements for fill fences and bulk heads.

2. A generally drier mine, which has intangible benefits for equipment maintenance and ventilation requirements

3. Less slimes to be cleaned and removed from the mine.

4. Faster curing rates for consolidated fill.

The net effect of a low water content hydraulic fill will be a stronger fill with lower binder required and an improved underground environment.

The technology involved in placing low water content fill is an improvement on systems already in place, with which the industry is familiar, so it is compatible with existing mining methods. The basis of the technology is the realization that high density slurries can be made to flow long distances through pipes without the need for excessive pressures if the size distribution of the fill is carefully engineered and controlled. The size distribution that allows a high density slurry to flow is also the size distribution which is optimal for strength development in the fill.

6.2.3 A Literature Review of Development to Date

The concept of paste fill in underground mines evolved from concrete pumping technology, where mixes containing fine and coarse aggregate with cement have been successfully pumped to heights exceeding 400 meters at outputs of over 120 cubic meters per hour (Anon 1988a, Berkovitch 1985). When the concrete mix has been optimized, pumping, horizontal distances in excess of 1000 m at rates of 35 cubic meters per hour have been reported at solids concentrations of over 90 percent by weight (Renetzeder 1988). Although concrete pumping is usually in an upward direction, whereas paste fill is transported downward or horizontally, much of the same technology can be applied to the pumping of paste backfill. In fact, the first pump manufacturers to be involved in high density underground backfill projects were considered to be experts in concrete pumping prior to their initiation into mining (e.g. Putzmeister and Schwing in Europe).

Paste fill is discussed as the ultimate in backfill technology in the literature, however in practice high density hydraulic fills seem to be more common. True paste systems are still not common in the mining industry. There have been a number of operations start out using paste systems but because of practical problems revert to a high density hydraulic fill with slightly more water added. This is possibly a consequence of the lack of suitable material for paste filling at most mines. The concrete industry from whom the paste ideas came from, pump slurries which contain significant coarse aggregate. In the mining industry this would be classified as a dense paste fill. The most successful paste fill operations appear to place fill which is similar to concrete, i.e. a dense paste fill with coarse aggregate.

The first mine to successfully apply paste fill technology was the Preussag lead-zinc mine in Grund Germany. With the financial support of the Federal German Government development of the method began in 1978 and was essentially completed in 1983 (Lerche and Renetzeder 1984). The Preussag system is known as "pumped fill" because it relies upon the application of positive displacement pumps. This pumped fill is a dense paste fill containing aggregate. It is pumped at 88 percent solids by weight. It has been pumped 500 m down and 1500 m across in a daily routine operation (Renetzeder 1988). Four-cylinder Putzmeister pumps are used. The Preussag Bad Grund system is documented in many publications. Ruehe & Kaemmerer (1988), and Renetzeder (1988) offer the most recent accounts of the

Grund Mine system. After the reported success at Grund Mine, similar efforts followed in many other countries, especially in South Africa where new methods of backfilling in very deep mines had been the focus of much research (see Verkerk 1982, Verkerk 1983, Verkerk 1984 and Verkerk 1986).

Several South African companies and the University of Witwatersrand through the Materials Handling Research Unit have been active in research and development of both paste and high density hydraulic fill technology. The Anglo American Corporation has placed paste fill at over 80% solids with the use of a Joy tailspinner centrifuge (Patchet, 1978). Western Areas Gold Mining Co Ltd has been reported as placing a pumped paste fill (combination of minus 25 mm run of mine waste, mill tailings and pulverized fly ash), (Clark, 1982, Barrett et. al., 1986). Randfontein Estates Gold Mines report the use of a pumped paste fill with dry binder added underground just prior to placement (Klintworth, undated). High density hydraulic fill is reported at the Black Mountain and Carolusburg mines in South Africa (Barrett et al, 1986). The Tsumeb mine in Namibia is reported to have placed pumped paste fill at 81% solids in cut and fill stopes (Barrett et al, 1986). Akouta mine, Niger, is reported to have placed low water sand aggregate fill with a slump of 4 to 6 cm through 200 mm boreholes to underground pumps which delivered the fill at 75 m^3/hr at 100 bar to the stopes (Bibert, 1982)

In Australia the Elura mine had a successful pumped a dense paste fill system operating for some time (now closed). Mount Isa Mines have experimented with Pb, Zn, Cu tailings fill at 76% using viscosity modifiers (Snow and Horsley, 1987).

The British Coal Corporation has been active in developing high density paste and hydraulic fills for packing in collieries. This system is based on coal washery coarse rejects suspended in a bentonite slurry.

The Lucky Friday Mine of the Hecla Mining Company Limited in Wallace, Idaho, USA has had a high density hydraulic backfill system, (Falconbridge internal report). The fill was placed at 82% solids by weight at a rate of 170 to 200 tph. The Cannon mine, USA, has been involved with the USBM in a "paste backfill pumping and design fundamentals" project and has employed a tailings paste fill with low cement and low slump (Corp and Skinner, 1989). Trials of a low water fill have been reported for use in cut and fill at the Sunshine mine, Coeur d'Alene, Idaho (Friel et al 1984).

In Canada, research on high density backfill was reported at INCO Ltd.'s Mines Research Laboratory in Sudbury, Canada (Landriault & Goard 1987, Stocks & Hodinott 1987). Pit run sand screened to 1 inch was transported in pipeline test loops at a concentration of 83 percent solids by weight using concrete pumping technology. A full scale trial of high density cemented sandfill was conducted at INCO's McCreedy West Mine in 1988 (Draper 1989). The trial indicated that paste fill was 2 to 5 times stronger than an equivalent hydraulic fill with similar binder content. Falconbridge Limited Sudbury Operations has conducted surface pump loop tests. The Dome Mine in Timmins has operated a tailspinner centrifuge underground to produce paste fill for backfilling old stopes (Churcher, 1990). Some initial investigations for this work at the Dome mine is reported in a CANMET project report, "State of the Art Review of Dewatering Schemes for Mine Backfill" (Project number 5-9204)

Other important research was carried out under the Canada Ontario Mineral Development Agreement, "COMDA". Three reports from this work are as follows: "In situ Determination of Dewatered Tailing Fill Properties in Ontario Mines. Supporting Date" (Project number 142502); "Lique Faction Potential of Dense Backfill in Ontario Mines" (DSS File No. 1SSQ23440-6-9039)"; and finally "In Situ Determination of Dewatered Tailings Fill Properties in Ontario Mines" (DSS File No. 0559-23440-J-9204).

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The list of research projects and applications of low water content backfills mentioned above is not complete, however it does indicate the level of interest in this new technology throughout the mining industry.

6.2.4 Paste Fill

6.2.4.1 Preparation

The two options for preparation of paste fill are:

1. Surface preparation of the paste with pipeline reticulation to underground, often with pumping to assist horizontal transport. Binder may be added at the surface plant or transported separately and added just prior to placement.

2. Underground preparation with a dewatering machine. The fill is delivered to underground as a typical hydraulic slurry, a dewatering machine will separate the solids from the water, producing the paste fill. The excess water containing residual solids is pumped back to surface or to a conventional hydraulic fill location. The dewatering machine may be located at the stope

to be filled so no subsequent transport is required or at a central location from which the paste produced is pumped to the placement location. Again, the location for addition of binder is flexible, but commonly it would be added at the dewatering machine.

For surface preparation there are many dewatering machines available, used in surface mills, sewage and other applications. The physical constraints of mine openings limit the number of machines suitable for underground paste fill preparation. The first underground dewatering machine used for fill applications was the "Tailspinner"; a centrifuge machine developed by Joy Manufacturers Co. (Wayment 1978). It has been primarily used in South African gold mines (Patchet and Currie 1982). The Dome Mine in Timmins, Ontario Canada has also used the machine to produce a paste fill at 85% solids by weight. Mill tailings in a 60% slurry were delivered to the machine at 36 tonnes per hour, output of paste fill was 11 to 13 dry tonnes per hour (Bharti et. al. 1989). An underground version of a rake classifier has been developed and used underground at the Cantung Mine in the Northwest Territories of Canada (Kowal 1983).The output was 85% to 88% solids and approximately 600 tonnes was placed per day. The South African gold mines have drum filter dewatering systems operating underground capable of higher production rates. Drum filter arrangements tend to be located at a central underground location from which the pastes produced are pumped to the stopes.

The principal disadvantage of underground dewatering is the need to handle the reject transport water, which can contain up to 40% solids by weight (Falconbridge Limited, 1989). Another problem is the relatively low capacity of the systems. The South African mines have been able to improve capacity by installing larger drum filter systems, but the capacity is still limited. The practical maintenance problems associated with having dewatering machines underground are not to be forgotten.

Underground dewatering systems can be used to produce a paste fill which is denser than that possible with surface preparation when the paste can be directly placed and does not have to flow any distance in a pipeline.

Higher capacity paste filling systems are for practical purposes restricted to surface preparation. Here conventional dewatering techniques and equipment can be employed to prepare the paste. Vacuum belt and drum filters are commonly used, with the belt filter being more popular. If the fill material is available in a dry form it may not be necessary to use expensive dewatering equipment. Mixing the paste to achieve a consistent density is given a considerable attention in the preparation plant. Maintaining a consistent moisture content in the paste is the most difficult aspect of the preparation of a low water content fill.

Mixing the high viscosity paste to achieve a consistent product is important. The paste fill particles must be uniformly wetted before being pumped. Screw mixers and pug mills (similar to those used in concrete preparation) are used for mixing.

The principal decision which must be made for a surface preparation system is when to add the binder. If added on surface there is potential for the paste to set in the distribution line, while if added underground close to the stope, a separate transport network for dry binder is required.

The flow sheet for paste fill preparation and distribution at the Bad Grund Mine, Germany is given in figure 6.1 and 6.2. In this system the binder is added close to the discharge point underground, capacity is around 30m^3/hr. The Elura Mine paste fill preparation plant flow sheet is shown in figure 6.3. In this system the binder is added on surface, (system no longer operational).

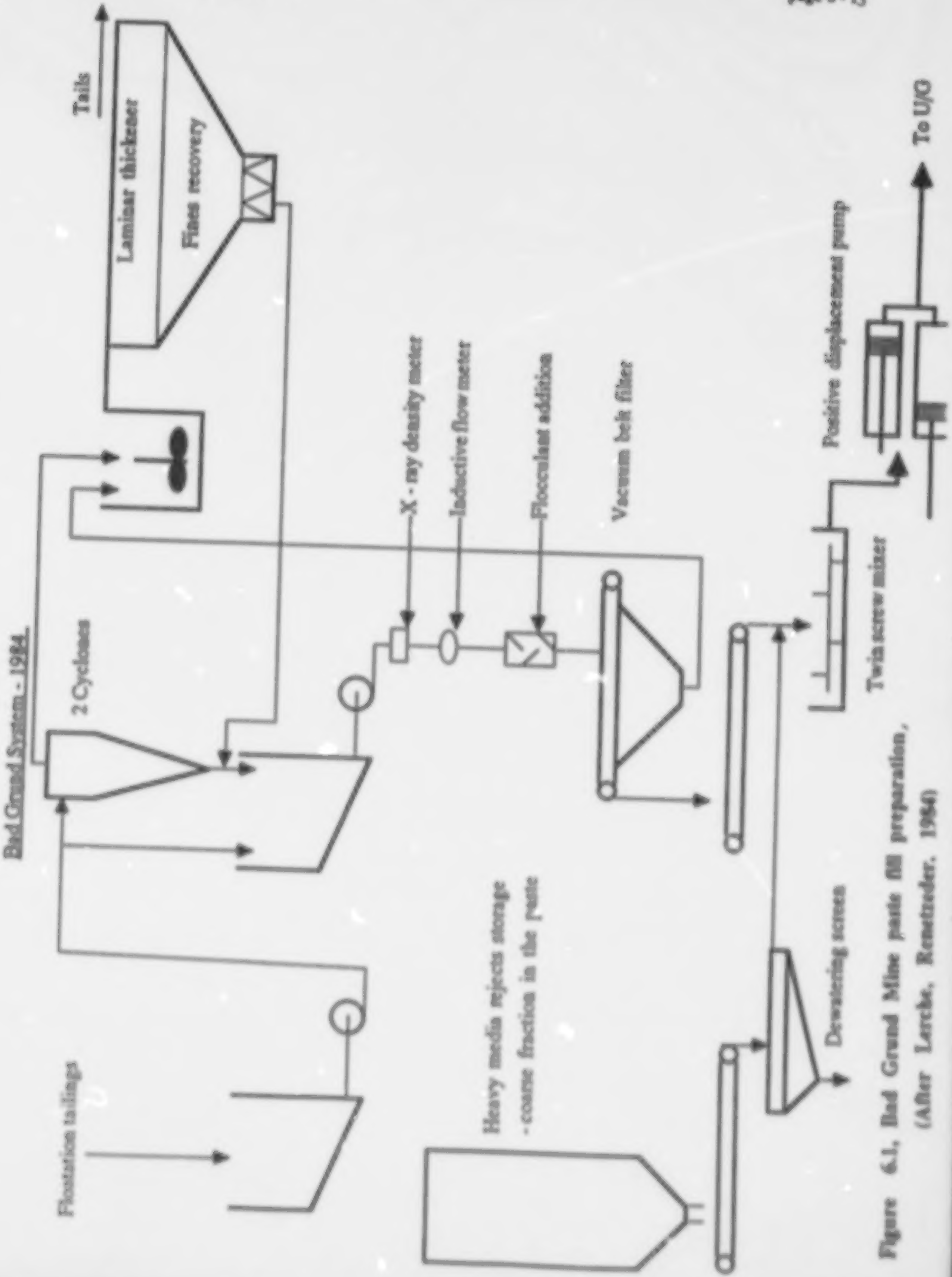

Tails
Laminar thickener
Fines recovery
Bad Grund System - 1984
2 Cyclones
X - ray density meter
Inductive flow meter
Flocculant addition
Vacuum belt filter
Positive displacement pump
To U/G
Twin screw mixer
Flotation tailings
Heavy media rejects storage
- coarse fraction in the paste
Dewatering screen

Figure 6.1, Bad Grund Mine paste fill preparation,
(After Lerche, Renetzeder, 1984)

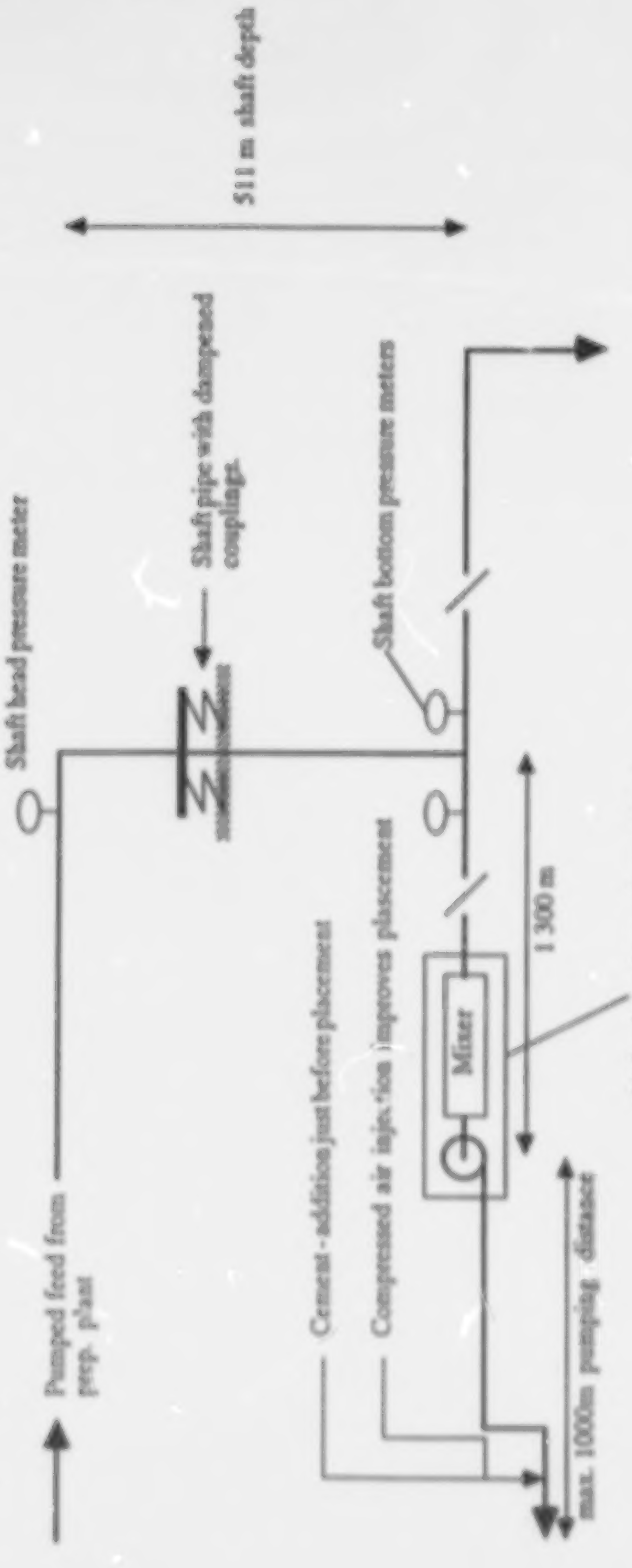

511 m shaft depth
Shaft head pressure meter
Shaft pipe with dampened couplings.
Shaft bottom pressure meters
Pumped feed from prep. plant
Cement - addition just before placement
Compressed air injection improves placement
Mixer
1 300 m
U/G mix and pumping station
(Pos. disp. pump)
max. 1000m pumping distance

Figure 6.2, Bad Grund mine paste fill distribution system.

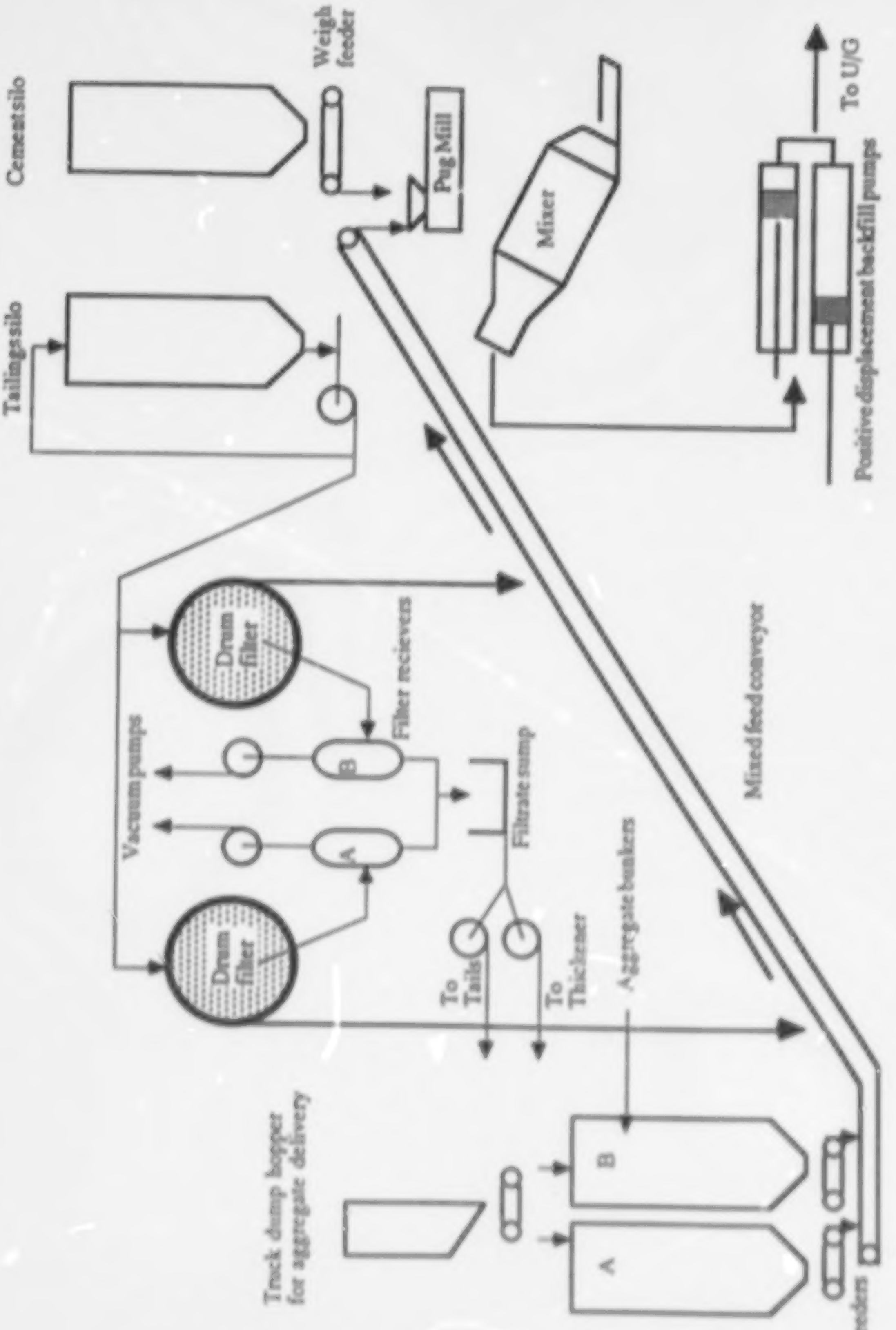

Cement silo
Weigh feeder
Pug Mill
Tailings silo
Mixer
To U/G
Positive displacement backfill pumps
Drum filter
Drum filter
Vacuum pumps
Filter recievers
A
B
Filtrate sump
To Tails
To Thickener
Mixed feed conveyor
Aggregate bunkers
Truck dump hopper for aggregate delivery
A
B
Feeders

Figure 6.3, Elura Mine backfill plant flow diagram,
(After Barrett et al, 1984)

6.2.4.2 Paste Fill Rheology

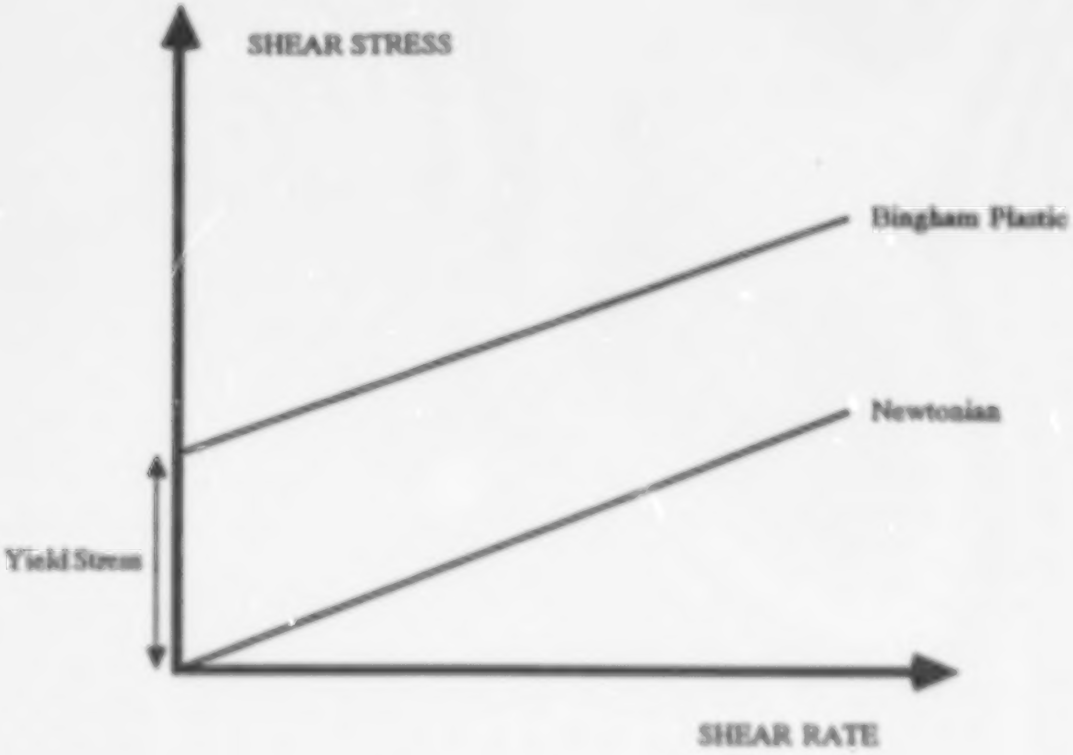
Understanding the rheology of a paste fill is critical when it must be transported over any horizontal distance by pipe. Paste fills have complex rheologies. It is not a simple matter to determine the engineering properties of a paste fill.

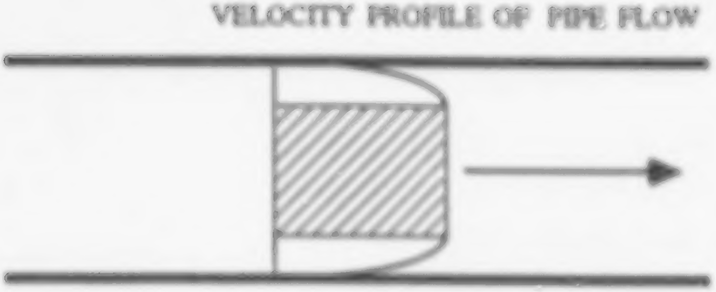
A paste fill will generally exhibit non Newtonian fluid characteristics, i.e. its viscosity will be dependent upon the shear rate (velocity of flow and pipe diameter) and possibly the length of time shear has taken place, as well as the temperature. In contrast the viscosity of a Newtonian fluid is dependent only upon its temperature. A conventional hydraulic fill, when sufficiently agitated, is considered to be a Newtonian fluid.

Paste fill rheology can be approximately described as Bingham plastic, or plug like flow, figure 6.4. Flow is of a solid plug of unsheared material in the center of the pipe, lubricated by a thin layer of less viscous material at the pipe wall. All shear takes place in a thin layer at the pipe wall. The high shear stress in this thin layer results in lower viscosities (a consequence of non Newtonian rheology) which helps to smooth the flow of the central plug of material.

A paste fill will generally have the properties of a thixotropic substance because it has a viscosity that is dependent upon the immediately prior shear rate and it has a tendency to take a set when at rest. In practice this means that the faster paste is mixed (high shear rate) the lower will be its viscosity. This has some important implications for mixing and pump feeding arrangements in handling paste fills as well as assisting pipe flow.

The difference between Newtonian and Bingham plastic rheology is shown in figure 6.4, a rheological diagram. This indicates that at a particular shear rate (velocity of flow) the shear stress required (line pressure) is much greater for a Bingham plastic than for a Newtonian fluid. This is a function of the greater viscosity of a paste fill. Also of interest is the initial shear stress which must be overcome to start the material flowing, which has implications for starting flow in pipelines full of stationary paste.




VELOCITY PROFILE OF PIPE FLOW

Bingham plastic plug flow -
all shear takes place at the pipe walls

Figure 6.4, Characteristics of Bingham Plastic flow, typical rheology of paste fill.

The practical conclusions from this discussion of paste rheology are that; paste fill flow is completely different to conventional slurry flow, high densities of paste slurries are not necessarily reflected in high resistance to flow. Engineering a paste fill to achieve an acceptable rheology is a balancing act between;

1. maintaining a viscosity high enough so that fill particles will be non settling at zero or very low flow velocities, i.e. essentially no critical velocity.

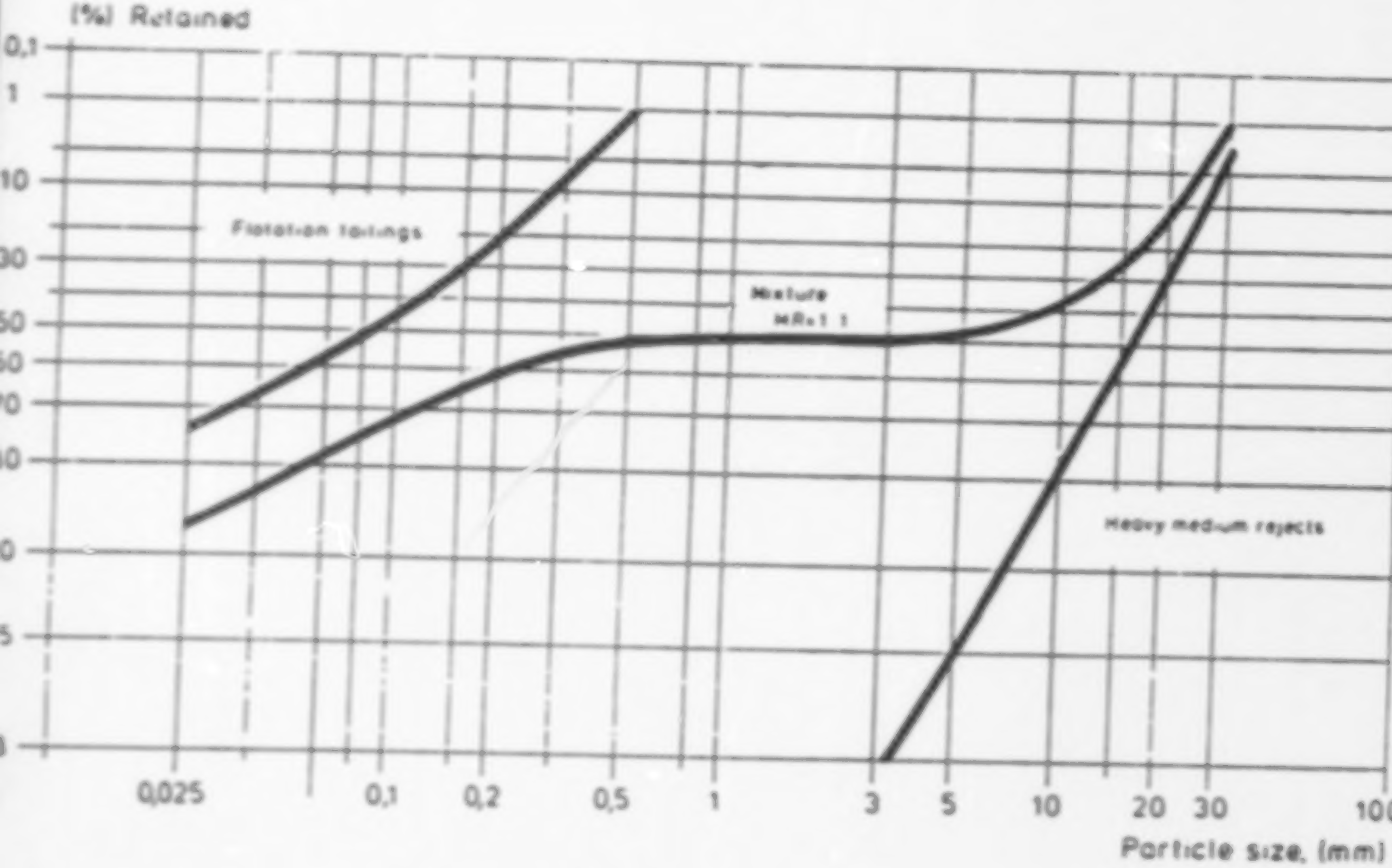
2. maintaining a viscosity low enough so that resistance to flow is not excessive.

6.2.4.3 Particle Sizing

The flow of a paste fill is characteristic of pumped concrete from which the concept evolved. Many of the pumped concrete design principles apply to paste fill, although the difference in particle sizes involved forces some changes. In pumped concrete the basic concept is to make a paste of sand cement and water which is of sufficient volume to fill the voids between the coarser aggregate material. The fines paste must have sufficient viscosity to maintain the coarse aggregate in suspension but not such a high viscosity that resistance to flow is excessive. The fines paste will lubricate the flow of coarse particles which it carries in suspension. Under pressure fines will migrate to the low flow regions immediately adjacent to the side walls of the pipe. Here they are subject to high shear rates and act to smooth the flow of the central plug of concrete.

Pumped concrete is based upon a bimodal size distribution. A fine size fraction which makes up the supporting paste, and the coarse aggregate fraction. A paste fill which matches this concrete type sizing is classified as a dense paste fill by definitions given earlier in this section. A simple paste fill is composed of fine material only. It is a crude generalization, but a fair comment to say that the most successful applications of paste fill have been those that resemble pumped concrete with a bimodal size distribution, e.g. a dense paste fill. Figure 6.5 shows the size distribution of the successful Bad Grund paste fill, which is a dense paste fill.

The minimum water content of a paste fill is a function of several variables, but particle size is one of the more important controls. Material with a fine particle size has a large surface area which must be wetted. This implies the need for a high water content. Introducing coarse material will reduce the surface area to be wetted and hence the water content may be reduced without affecting the viscosity. A paste fill without a coarse component will in general be placed at a lower solids concentration, or a higher viscosity than a paste fill which has coarse material included.



PREUSSAG

Figure 6.5, Size distribution of Bad Grund dense paste fill, a mixture of floatation tailings with coarse heavy media rejects, (After Lerche, Renetzeder, 1984).

The edition of fine clay or bentonite has been used to improve the flow characteristics of simple paste fills. The clay particles form a lubricating layer in which the coarser sand or tailings particles are suspended.

The volume of paste formed by the fines, water and cement in relation to the volume of voids in the coarse material is critically important. The volume of paste must exceed the volume of voids so that all voids between the coarse particles are filled to produce a flowable slurry for pumped pipeline transport (Remetzeder 1988).

The maximum particle diameter which can be carried efficiently is 40 percent of the pipeline diameter (Barrett et al, 1986).

6.2.4.4 Delivery and Placement

The high viscosity of paste fill means that it has a high resistance to flow. It is commonly transported at low velocities to compensate for this. Typical velocities are 0.3 to 1.0 m/s. Such low velocities are possible because a paste has no critical velocity; it is designed to be non settling at least for limited periods. A paste will flow in the laminar regime.

Pressure loss rate in transport of a paste fill is dependent upon flow velocity, viscosity, pipe diameter and the use of chemical flow modifiers. These factors are very specific to particular fill materials and size distributions. A few generalizations can be made about paste fill flow which are consistent with standard hydraulic experience;

1. Pressure loss increases rapidly with flow velocity. In practice the velocity is limited to about 1.0 m/s.

2. Pressure loss is significantly reduced by increasing the size of the pipe.

3. Reducing the viscosity of the paste reduces pressure loss significantly. Coarse aggregate may be added to keep overall density high while reducing the viscosity of the transporting fines paste and hence the overall pressure loss. Paste rheology is complex so that viscosity is affected by flow velocity; engineering these materials is an iterative process.

Typical pressure losses quoted for paste fills flowing at up to 1.0 m/s are in the range 4 to 22 kPa/m. The broad spread of values here reflects the wide range of paste fills and the number of factors which effect paste flow. For conventional hydraulic fill flowing at 3 to 5 m/s pressure loss in the range 2 to 4 kPa/m might be expected.

Pumpability can be improved by adding flow modifiers to the mix being pumped. For example, a small amount of sodium tripolyphosphate ($Na_5P_3O_{10}$) added to a slimes slurry containing about 75 percent by weight solids can reduce the pressure gradient by 55 percent, resulting in enhanced pumpability. A major problem with this additive is that the slurry must be neutralized before the backfill enters the stope. This is done either by adding chemical neutralizers or by making pH adjustments to the slurry. Details are given in Streat (1986) and also in Verkerk (1984) and Verkerk (1986). Various polymers added to fine slurries can also reduce the pressure gradient of a dense backfill system (Verkerk 1986).

Paste fill placement rates are restricted by the high pressure drops and practical constraints on pipe lines and pumping equipment. Pipe size is limited by what can be practically managed underground. Typical placement rates, using off the shelf pumping equipment, is about 50 m^3/hour. Higher placement rates are possible with parallel systems or special high pressure high capacity pumping equipment. These restricted placement rates are a major problem with the implementation of paste fills.

Pumps are frequently required for the transport of paste fills. The high pressures required and the high viscosity of the material make positive displacement pumps the most suitable choice. High pressure twin-cylinder reciprocating-type positive displacement pumps are recommended (Marcus et al 1982, Barrett et al 1980). These pumps are not only technically suitable for pumping paste, but they also offer easy maintenance and minimal air entrainment so that cavitation potential of the material is reduced (Barrett et al 1980). They are also sturdy, safe and economical to operate and are reliable under the most extreme site conditions. One large pump manufacturer, Putzmeister has introduced a four-cylinder pump for pumping backfill at the Grund Mine in Germany (Renetzeder 1988, Ruehe & Kaemmerer 1988). The four pistons ensure a continuous flow of material through the pipeline with very little stagnant time between strokes.

The pump system incorporates a receiving hopper into which the material is poured from a mixer or a site batching plant. The receiving hopper can contain an agitator to re-mix the material being pumped, but a re-mixer is recommended only for stiff mixes (Anon 1986). Screw feeders can also be employed to help inject very stiff mixes into the pump. Other components of the pump system include valves for isolating the material in the pumping chamber from either the delivery pipeline or the receiving hopper, and a system (usually hydraulic) to transfer motive power to the moving parts of the pump.

Standard high pressure ductile iron or steel pipes are used for the transport of paste fills. The pipe strength is restricted by the weight of pipe which can be handled underground. Pipe coupling that ensures a smooth internal surface is considered critical to the transport of paste, as it is in pumped concrete. Obstructions in the line (e.g. poorly coupled pipe) may cause segregation of the aggregate from its supporting paste and subsequently, line blockage. The standard victaulic pipe coupling systems used in mines for water and conventional fill lines may not be sufficiently rigid nor tight fitting for paste fill distribution.

Whenever new pipes are being used, it must be remembered that the internal surface of those pipes will be relatively rough when compared to pipes which have been used for some time (Loadwick 1970). In "wearing in" a new pipeline, pressures about 30 percent in excess of those predicted by design calculations may initially be required for pumping. As the medium being pumped scours the inside of the pipe, the excess pressure required is reduced.

Prior to beginning the paste filling operation, the pipeline must be thoroughly wetted. If this procedure is not carried out, the first fill through the pipe acts as the wetting agent, resulting in drying out of the paste, so that the mixture may become too dry to pump (Klintworth undated). The wetting operation usually involves pumping clean water ahead of the backfill paste. The water and paste are separated by two "pigs" (Klintworth undated). The paste is then directed through the pipelines to the stope to be filled. This procedure is also recommended by Verkerk (1984) of South Africa, who adds that a dilute slurry pumped ahead of the high concentration paste can be used as a flushing agent instead of clean water, but that the two should still be separated by "pigs" or sponge balls to help in diverting the dilute slurry away from the stopes to be filled.

Once the stope is completely filled, the pipelines which contained cemented backfill paste must be flushed out. This is again accomplished by pumping clean water down the lines behind a "pig" which separates the fill from the water. It is important that the flushing water is not dumped in the recently filled stope. It is possible to design a paste such that pipelines or boreholes which contain unconsolidated paste without binder may be left full for extended periods, hours or days.

Pipe wear with paste fill is moderate due to the low transport velocities.

Paste fills do not flow readily and so the method of their placement must be different from traditional hydraulic fill practices. A paste will typically have an angle of repose on placement of 35° to 65°, depending upon the mix. This is a major advantage because it means there is little or no requirement for fill containment barricades. Construction of fill containment barricades for conventional hydraulic fill is expensive labour intensive work. Operations placing paste fill may sometimes use simple barricades or waste rock walls if some containment is required.

The high angle of repose is however a problem when it comes to spreading the fill. Multiple pour points, compressed air injection, and slinger conveyor arrangements have been used to help spread paste fill. Sequences for pouring and placement of paste mixes follow a logical pattern. In areas which are accessible, work should be arranged so that fill is first placed at the furthest point of pouring (Berkovitch 1985). As the pour continue back towards the pump, sections of pipe are progressively removed. This sequence helps prevent blockages. Injecting compressed air at the end of the placement pipeline to impart momentum to the fill material and ensure dense and uniform placement of the fill in the stope is a common practice, (Verkerk 1984).

Before designing a plant to produce high density backfill, laboratory and pilot plant studies must be conducted. These will determine values for the design parameters, such as of pressure losses in pipes at different inclinations (horizontal to vertical), settling velocity of the material, thickest slurry that can be transported, etc. (Rai 1974). When this has been completed, the next step is full scale testing to verify the previous parameters prior to developing a permanent installation. Methods for the scaling-up of test data to help design a full-scale test plant are poorly developed at present, and scale-up can be a problem due to the lack of theoretical information available. For limited information on scaling-up of test data, the reader can refer to Kazanskij (1978), Kazanskij (1976), Webster and Sauerman (1978), Sauerman (1983), Eyler and Lombardo (1982) and Sabbagha (1982).

6.2.4.5 Addition of Binder to Paste Fill

A binder can be added to high density backfill either on surface or underground. There are benefits and disadvantages associated with each of these options. Adding the binder underground requires a separate transport system to convey it to the underground mixing site. It also requires that a comparatively elaborate mixing and batching system be installed underground. When the backfill arrives underground with binder already included, the only requirement is for a re-mixer, re-pulper or feeder to provide the proper feed to any pumps, otherwise nothing. However adding binders on surface results in a higher risk of blockage of pipes in the event of an emergency shut-down. Residence time of an unconsolidated material in the pipeline can be considerable (dependent upon paste design, some have no residence time), whereas consolidated material may remain motionless for only a few hours before plugging of the pipe will occur due to binder activation. Chemical retarders may be added to control the binder curing rate a little. Adding the binder on surface also means that lines must be thoroughly flushed immediately after each use so that no build-up or plugging occurs.

6.2.5 High Density Hydraulic Fill

High density hydraulic fill is an intermediate fill type; lying somewhere between conventional hydraulic fill and paste fill types. It is characterized by a higher pulp density than conventional hydraulic fill, but still flows in the turbulent flow regime and has some excess water to drain, but permeability need not be a primary concern.

The distinction between a paste and a dense hydraulic fill is a little grey. Probably the best way to define a paste fill is that it will exhibit a measurable slump, in a similar fashion to concrete. A dense hydraulic fill is sufficiently fluid that slump has no meaning. The other principal distinction is that, like a conventional hydraulic fill, a dense fill will have a critical velocity. A dense hydraulic fill is a settling suspension, a paste fill is non settling (at least for a limited time). A dense hydraulic fill will look and perform very much like a conventional fill. The distinction between a dense and a conventional hydraulic fill is very difficult to define and is usually a subjective decision. A dense fill is really just a conventional fill with minimum excess transport water.

6.2.5.1 High Density Hydraulic Fill Preparation

The preparation of a high density hydraulic fill is much simpler than for a paste fill. It is similar to the preparation required for conventional hydraulic fill except that a supply of fill material, tailings or sand must be available in a dry state, e.g moisture content lower than that achieved with settled storage. The control of moisture content is the most critical parameter in the preparation of low water content fills and often the most difficult to control. The viscosity of high density hydraulic fills is not so high that special mixers or positive displacement pumps are required, so the preparation flow sheet will closely resemble that of a conventional hydraulic fill plant where dry materials are handled.

6.2.5.2 Rheology

Dense fill rheology is essentially conventional Newtonian behavior. A relatively high viscosity and hence resistance to flow will be noticed because of the low water content. Figure 6.6 illustrates the theory behind dense fill transport; by changing the size characteristics of a hydraulic fill (adding more fine sizes) the critical velocity at high density can be maintained at a reasonable level and may even drop. This will allow the transport of a high density fill in a pipe system similar to that used for conventional hydraulic fill at similar velocities. The increased resistance to flow due the higher viscosity will limit the horizontal distance which a dense fill can be transported with a given vertical head and with given pipe strengths compared to those achievable with conventional hydraulic fill.

The engineering design requirements for dense fill should be based on the horizontal transport distance. An operation with significant vertical head and only limited horizontal transport needs can place a relatively unsophisticated fill with very low water content because the transport characteristics are unimportant. There have been several examples of simple paste or concrete like backfills being placed by dumping directly via a vertical bore hole straight into the void to be filled. High density hydraulic fills of coarse classified mill tailings are regularly placed in operation that have only limited horizontal transport distances. As soon as the fill is required to travel further along on the horizontal the amount of engineering required to develop a workable fill mixture with the right sizing increases exponentially. Most reported dense fill systems to date have been those with good vertical head and limited horizontal in the distribution system.

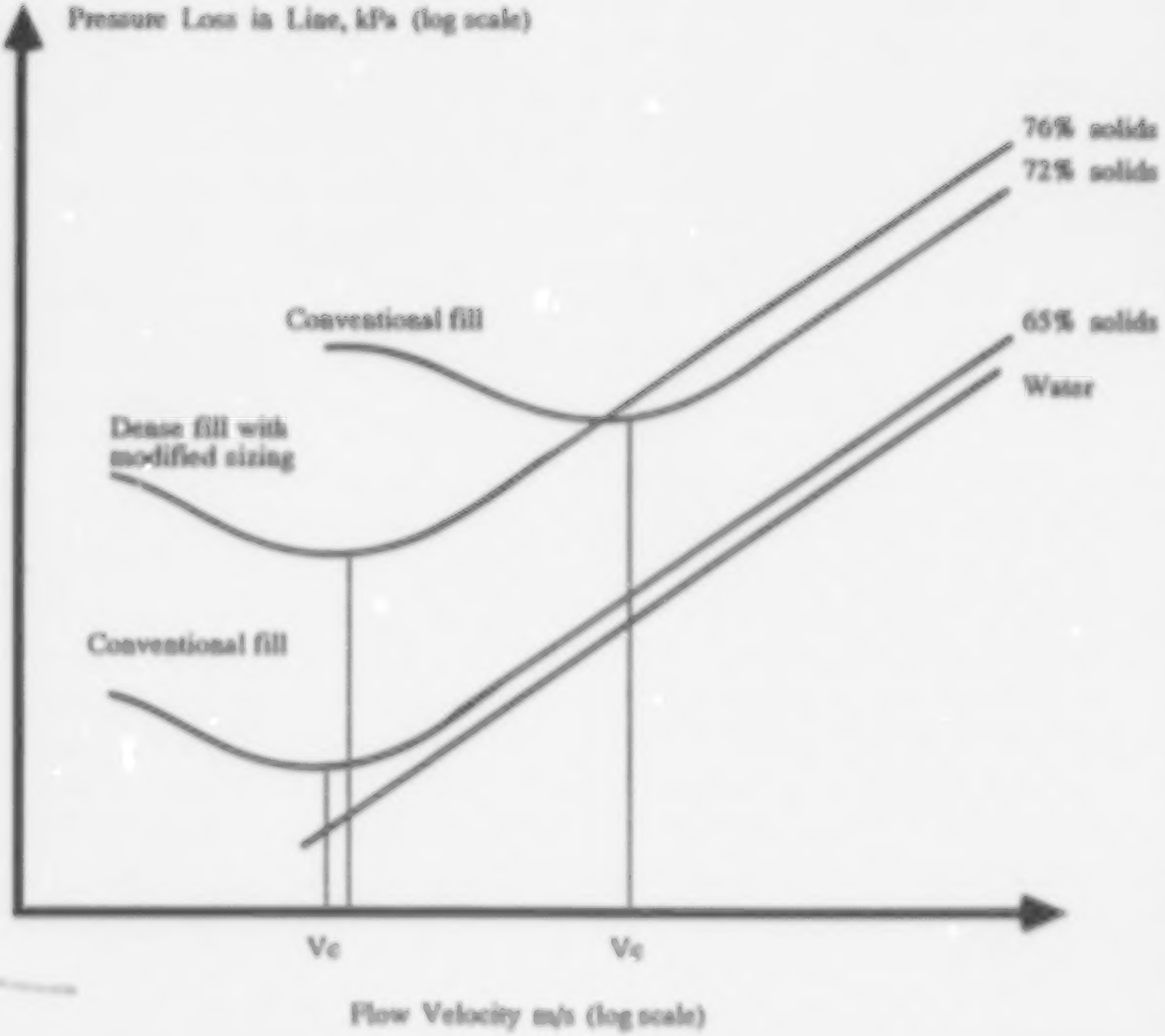
6.2.5.3 Particle Sizing

There are no hard rules which govern the particle sizing for a dense fill. As mentioned above the horizontal transport requirements will dictate how much size modification will be required to produce a viable dense fill for the particular distribution network. Engineering the particle sizing is aimed at modifying the viscosity characteristics of the fill.

It is well known from conventional hydraulic fill operations that increasing the solids concentration of a fill will increase the viscosity. This is limiting factor on the solids concentration that can be transported. A dense fill is possible if the solids concentration can be increased faster than the viscosity. In general this will require changing the particle sizing of the fill by increasing the proportion of fines present. This is a similar theory to that applied in paste fill technology except in dense fill the objective is a very well graded material rather than the bimodal sizing of a paste.

6.2.6 Permeability and Liquefaction

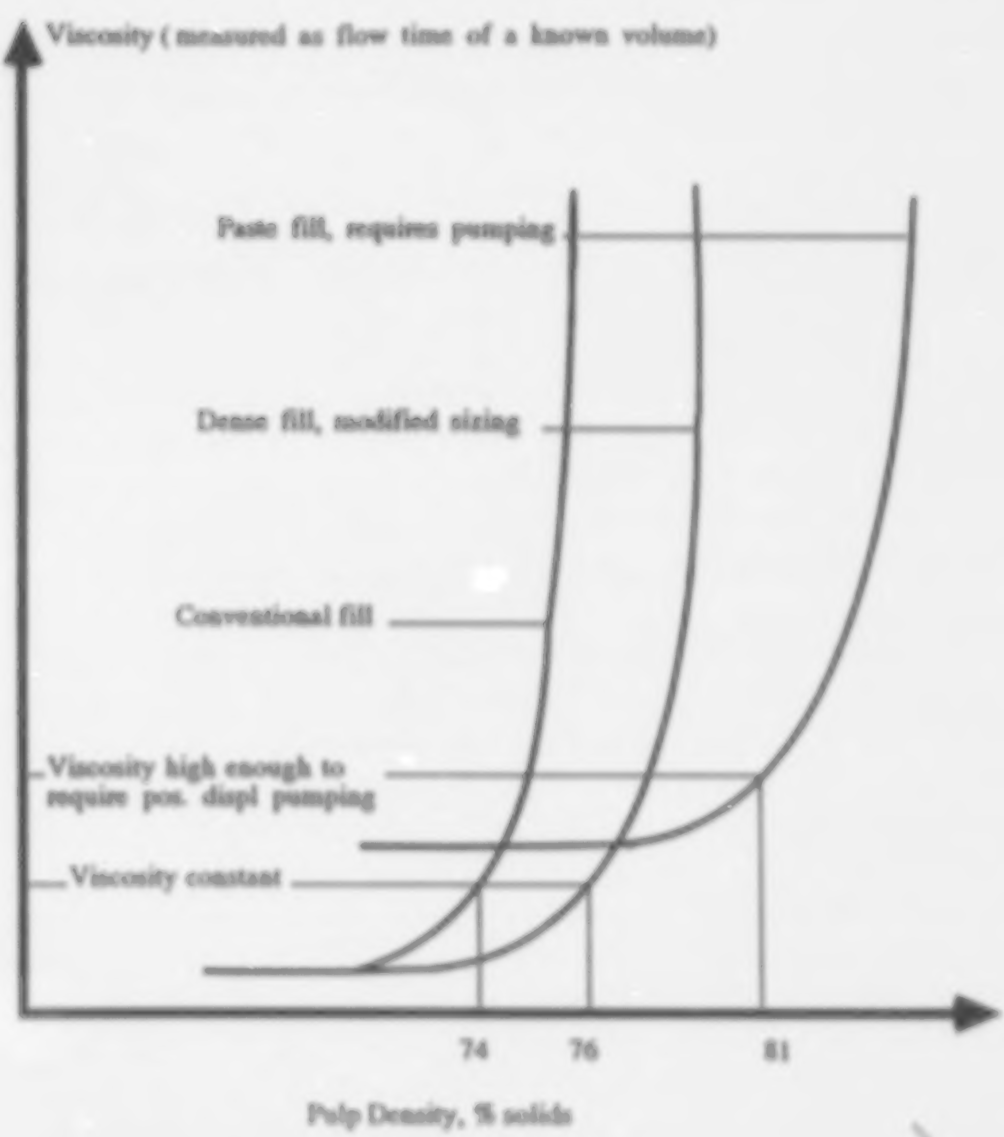
The effect of additional fines content in dense and paste fill is to significantly reduce the fill permeability. This has some major implications for such fills which are placed, without binder, and which may be subject to liquefaction. The containment of such fills with adequate barricades should be carefully considered by operations contemplating application of these technologies. The usual solution is to place only consolidated fill with some cement content to preclude the chance of any liquefaction. This may alter the economics of paste or dense fill systems, although their principal economic advantages are realized with consolidated fills any way. It is outside the scope of the report to get involved in a detailed discussion of the problems of placing unconsolidated dense or paste fill.



If we modify the size distribution of the fill by adding more fine sizes then the critical velocity at high solids concentration can be kept low

A dense fill will flow in normal pipe with only a marginal increase in pressure loss at normal velocities.

Figure 6.6, Dense hydraulic fill transport theory.



By modifying the size distribution of a fill we can keep the viscosity constant while increasing pulp density.

Figure 6.7, Viscosity characteristics of dense hydraulic and paste fill.

6.3 PNEUMATIC BACKFILL

Pneumatic backfill is a minor backfill method in the underground hard rock mining industry, but a discussion of its theory and practice is presented here for completeness.

6.3.1 Background

The first application of pneumatic backfilling, also called pneumatic stowing, dates back to the 1920's when European coal mines employed compressed air to convey 3-inch sized pieces of soft rock through a pipeline to stabilize underground chambers (Ball & Powell 1972). More recently, pneumatic backfilling has been employed in coal mines in the U.S. and in mines located in regions where water supplies are scarce. Pneumatic filling has also been popular in the U.S. for building seals in abandoned mine openings and old workings. Here, crushed limestone of 38 mm diameter or less is conveyed with or without portland cement, expansive cement and/or bentonite. This sealing technique has been found to be safer, faster and more economical than conventional methods (Maksimovic & Draper 1982, Brezovec 1983). Capacities achieved in pneumatic mine sealing were reported to range from 15 to 50 tph (US tons) (Roberts & Mamillo 1986).

Pneumatic backfilling in operating hard rock mines has its origins in Canada. In 1966, Cominco's Sullivan Mine in Kimberley B.C. teamed up with Radmark Engineering to develop a pneumatic backfill system capable of withstanding the high abrasion associated with hard rock stowing. The objective was to backfill large blasthole stopes using pneumatic conveying methods. The results of their efforts are reported in Ball (1970), Ball and Powell (1972), Reynolds (1972) and Soderberg and Corson (1976). By 1968, the equipment developed at Sullivan was capable of economically conveying hard, abrasive igneous rocks up to 76 mm in diameter at capacities of up to 272 tonnes per hour and over a distance of 760 m (Ball & Powell 1972). More than 43 500 tonnes of fill were placed pneumatically in one of the blasthole stopes at Sullivan Mine. Data indicated that pneumatic filling had cost one third less than the conventional "scraping" technique used at the mine (Soderberg & Corson 1976).

The development of hard rock pneumatic stowing at Sullivan Mine led the way to several future uses of pneumatic backfilling in hard rock mines throughout the world. One such example is found at the Kerr McGee uranium mine in New Mexico, U.S.A. Here, a high capacity pneumatic backfilling installation based on the Canadian developed system was commissioned in 1971 (Ball & Powell 1972). This system was installed on surface for backfilling cut-and-fill stopes at a depth of 441 m (1450 feet) through boreholes to surface. The design allowed pneumatic transport of backfill along a 198 m horizontal length, a 488 m vertical length and through one 60 degree and four 90 degree bends. The system consisted of a completely self-contained trailer-mounted unit producing a capacity of 227 tonne per hour.The portability of the unit allowed service to several locations where boreholes to surface were made available. The fill material used was minus 20 mm desert sand. Pneumatic backfilling has been used at smaller mines to provide high quality fill. The small Apex mine in North America, has used pneumatic filling to place excellent quality (5MPa) fill that allowed undercut and fill mining in extremely poor ground. Placement rates were 23 tonne per hour, blowing crushed minus 25.4 mm diameter limestone and portland cement (Guill, R.W., 1990).

A recently described application of pneumatic filling is in localized subsidence control by spot-supporting in coal mines (Carlson & Saperstein 1989). Here, an artificial pillar of relatively small dimensions is placed pneumatically under a strategic surface structure for subsidence control. In many instances, this technique can provide as much support as full-face pneumatic stowing at a fraction of the cost. According to Carlson and Saperstein, localized subsidence control by spot-filling using pneumatic methods is only feasible when a high-value structure is threatened by trough-type subsidence.

6.3.2 Basic Operating Principles

The basic theory underlying pneumatic filling is that in order to move solid fragments through a pipeline, it is necessary that the velocity of conveying air be sufficiently high to keep the particles in suspension. In addition to air velocity, the length of the pipeline is also important. Studies indicate that for a given system power rating, the velocity of pneumatic fill increases with increasing pipeline length. Very early reports have dealt with length of pipe and quantity of air and power required for efficient pneumatic stowing (Peter 1952, Peter & Horturg 1961). Additional factors such as density achieved, velocity at placement, power consumption, machine wear and general technical and economic value are also important operating parameters (Soderberg & Corson 1976).

6.3.2.1 Particle size affects

Particle sizing affects a pneumatic filling operation in several ways. First, power requirements of the conveying system will vary with particle size for a given pipe length. Increased particle sizes require higher air velocities, making additional demands on power. Pipeline length is also a factor. The relationship between particle velocity and pipeline length for different sized particles is shown in figure 6.8. Also, large particles accelerate at a slower rate than small particles, meaning that large particles require a longer transport distance before reaching their full conveying velocity. This means that there will be a minimum pipe length required for a given aggregate size to achieve the output velocities required to produce a well-placed consolidated fill. The effect of particle size on required pipe length for different system capacities is shown in Figure 6.9.

Particle size also has a marked effect on segregation of fill material during placement. If the discharge operation is not controlled by using deflectors or by a sweeping action, coarse and fine material will segregate during placement. This can be explained by the fact that small particles lose velocity rapidly after discharge. Large particles tend to fall short, but they roll farthest due to their higher momentum. Medium sized particles are thrown farthest. Consequently, similar sized particles tend to congregate at their landing site, resulting in size segregation, unless outside measures are taken. It should be noted though that the free-throw distance of pneumatic backfill also depends on pipe layout and landing area restrictions.

Another effect of particle size is evident in wear rates of pneumatic system components. As the particle size is decreased, pipeline erosion decreases significantly. Also, larger particles are more prone to plugging in the pipe (Carlson & Saperstein 1989).

Specific design equations for pneumatic backfilling systems have been kept proprietary by the manufacturers of pneumatic conveyors, but they can be derived from any scientific publication dealing with powder technology and pneumatic transport related to other industries such as food and drug manufacturing. The application of pneumatic pipelines in the minerals industry has not received wide attention because of unfavorable material properties such as abrasiveness and high density, but pneumatic transport is used somewhat for coal, gypsum, some pulverized ores, alumina, cement and other materials (Anon. 1979a). It is stated by Soo (1984) that a horizontal distance of up to 4 miles is economically and technically possible using pneumatic transportation.

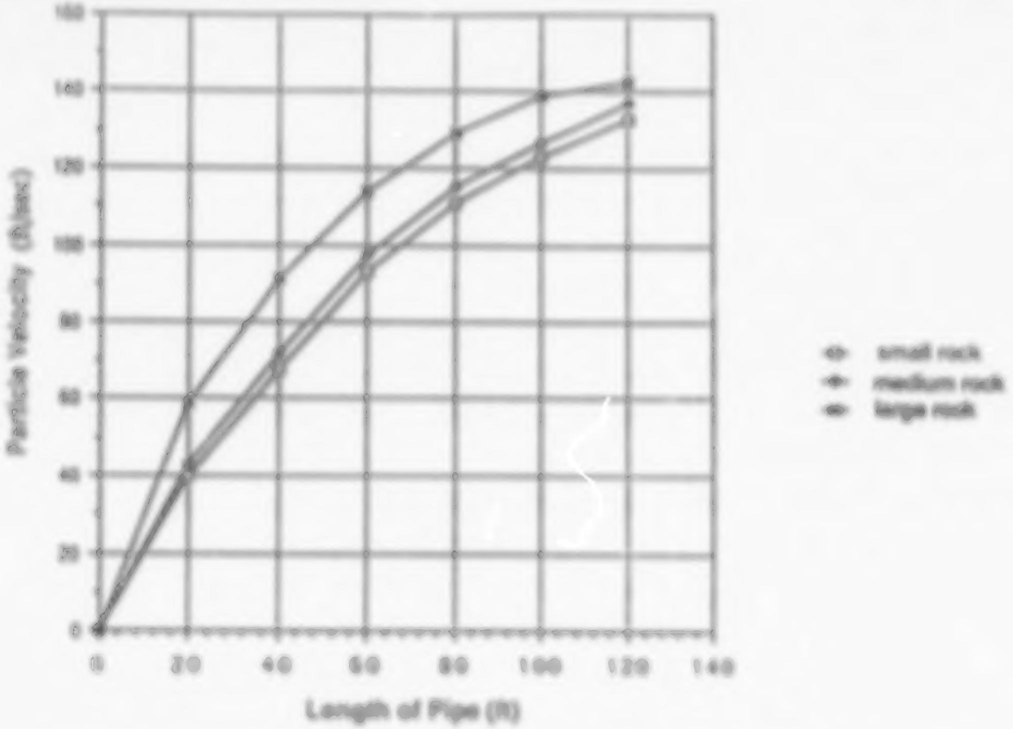


Figure 6.8, Effect of length of pipe on particle velocity for different sized particles, (After Ball, 1970)

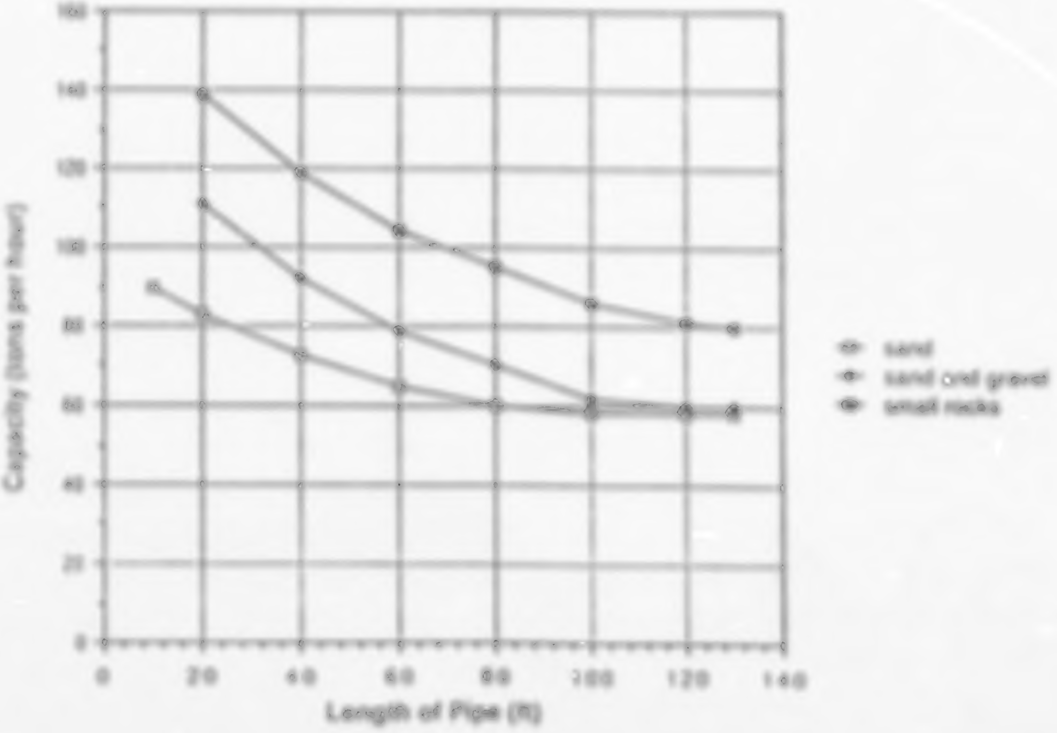


Figure 6.9, Effect of length of pipe on system capacity for different sized rocks, (After Ball, 1970)

6.2.4 Density and Consolidation of Pneumatic Fill

An attractive feature of pneumatic filling is the self-consolidation potential of the material upon placement. Densities of up to 80 percent those of solid rock can be achieved with optimized pneumatic placement of fill (Ball 1970), resulting in excellent support capabilities.

Several factors contribute to compaction of pneumatic fills, the most important being the high conveying velocity required to maintain the fill particles in suspension in the pipeline. This high velocity gives high impact forces to particles upon placement resulting in mechanical compaction. Higher discharge rates result in denser fills due to the greater velocities required. A critical length of straight pipe is required to allow sufficient time for particles to achieve their maximum velocity. It is therefore recommended that any bends required be placed as far away from the discharge end of the pipe as possible so that the particles will have time to re-attain that maximum velocity.

Since the impingement force of particles being discharged promotes compaction, the mechanics of the discharge action also affects density. It has been found that if a stream of material is aimed at one place for a long time, segregation can occur due to coarse material rebounding and rolling to the side or down slope (Soderberg & Corson 1976). This causes an accumulation of fines and a poorly compacted fill. The best densities can be achieved by either confining the area being filled to prevent rebound, or by using a deflector to provide a full sweep of the discharged material. This allows rebound material to be packed in with new material and gives a higher average density. Deflectors are hydraulically operated and can be automated for vertical and horizontal sweeping movement.

Density and compaction can also be improved by proper particle gradation. Although pneumatic conveying does place a limit on the coarseness of the material which can be transported, it is desirable to have a mix of fine and coarse fractions to optimize the density of the placed material. The coarse fraction should consist of material which is neither brittle nor friable so that particle attrition does not produce excessive fines. Generally, a well-graded mix with a high percentage of coarse material will have a higher density than a finer, well-graded mix for two reasons: a) a 76 mm piece of solid rock has less voids than the same volume of sand or 13 mm pieces, and b) the energy of smaller particles is insufficient to accomplish additional compaction, but if the kinetic energy of the large particles is greater than the shear strength of the previously placed material, compaction will occur (Soderberg & Corson 1976). In limited tests conducted by Soderberg and Corson, a high density fill could be produced using as much minus 76 mm material as possible, mixed with 13 mm well graded material and with just enough minus 13 mm particles to fill the voids between the larger material. This worked for the sand-gravel mix which was tested, but the values given will not necessarily apply to other types of backfill material. Tests with site specific materials should be conducted to determine optimum particle grading for a given application.

Water is often added to pneumatic filling systems as a means of suppressing dust. When water is added to the conveying line, it vaporizes and causes each individual particle to be wetted. This causes particles to stick together as a mass after placement through the action of capillary forces (Carlson & Saperstein 1989). Hence, water in pneumatic fill also encourages fill compaction, creating a higher density fill. Excessive amounts of water will cause slugging of the material in the delivery lines, and water running at the nozzle end while stowing. The optimum water content must be determined by performing a series of trial runs. When injecting water into a pneumatic system, it is sometimes preferable to add it close to the discharge end of the circuit rather than at the feeder unit so that transport loads are kept as low as possible.

Further consolidation of pneumatic backfill by adding a binder is also possible. Reynold (1972) stated that cement grout could be introduced into the feeder along with the fill in proportions of 1:30 cement to fill. The grout would then be transported pneumatically with the fill.

6.3.4 Pneumatic Filling Equipment

Basic pneumatic backfilling systems consist of three main components: the air source, the feeder and the pipeline. Detailed descriptions of pneumatic filling equipment are given in Ball (1970), Ball and Powell (1972), Reynolds (1972) and Soderberg and Corson (1976). Main features are summarized below.

Feeders can consist of either compressors or commercial blowers. If a constant volume of air at pressures below 138 KPa are desired, a blower can be used as the air source. If pressures above 138 KPa are required, a compressor must be used. The required operating pressure for a system increases with the length of pipeline, the number of bends, the vertical distance of lift, the frictional characteristics of the material conveyed and the volume of material being transported (Ball & Powell 1972).

The feeder consists of a rotating-pocket type airlock wheel which moves material from the low pressure air zone (i.e. atmosphere) to the high pressure air zone inside the conveying pipe. This type of feeder is suitable for hard abrasive material because it allows fine adjustments to be made to compensate for wear. The tips of the rotor are replaceable and are surfaced with abrasive-resistant tungsten-titanium alloy (Soderberg & Corson 1976, Ball & Powell 1972, Ball 1970). A schematic of a typical feeder which is suitable for handling igneous rock backfill is shown in Figure 6.10.

The pipeline used in pneumatic filling is usually of 203 mm (8 inch) inside diameter and is constructed of two layers. The inner layer ranges in thickness from 3 to 7mm and is extremely hard, having a Brinell number of 650 to 700 to combat wear. The outer layer is made of 3mm thick mild steel. It is very important that couplings employed allow the pipeline to be positively aligned to prevent obstacles to the air path which could cause friction losses or blockages. Bends represent areas of high wear and also cause losses in velocity. Therefore, their number must be kept as low as possible. Pipe bends should be manufactured with abrasive-resistant impact sections. As mentioned previously, the discharge end of the pipe should incorporate hydraulic deflectors to permit a sweeping path of fill to be discharged.

One recent improvement to pneumatic backfill discharge pipes involves a flexible telescopic pipeline which permits motions of up to 9.1 m at the nozzle without the need to dismantle and re-couple pipes (Carlson & Saperstein 1989). Also, the "lateral stowing" technique can improve discharge efficiency (Roberts & Marsillo 1986). This method employs a pipeline with multiple discharge points along its side. Hydraulically operated deflectors discharging perpendicular to the face are installed throughout the pipeline. Deflectors can be activated when needed to direct the flow to desired areas without dismantling pipes, resulting in lower labor costs.

6.3.5 Wear of Pneumatic Components

Wear of equipment components is a great problem in pneumatic backfilling. Wear is usually concentrated in certain sections of the circuit. In straight pipe sections, wear is less in the sections prior to the material reaching its peak velocity, except in areas of turbulence. Wear also occurs in a track along the bottom invert of pipes where larger particles tend to roll. Pipes should be rotated periodically to distribute wear evenly along the entire inner surface. Pipe joints and bends are also the site of severe wear which can sometimes lead to perforation of the inner pipe. Wear is also evident in valves (see Gotchow 1981 for a discussion of the different types of valves for use in pneumatic transport).

6.3.6 Advantages and Disadvantages of Pneumatic Backfilling

There are several advantages to using a pneumatic system for delivering backfill underground. The system is capable of conveying horizontally, vertically and around bends, making it desirable in locations where material handling equipment needs to be compact and portable. Fill can be got tighter to the back than with conventional techniques. When proper materials and controlled conditions are used, pneumatically placed material can produce a higher density fill than conventional methods normally used in mining. There is no excess water to decant or to be pumped out of the mine. There is also less spillage and less cleanup required on fill levels. Finally, no elaborate fill barricades are required because the fill is almost self supporting as soon as it is placed.

The *disadvantages* of pneumatic backfill are numerous. First, there is the problem of dust generation, resulting in a need for elaborate dust control systems at the inlet and discharge points. Another drawback is the excessive wear associated with pneumatic systems resulting in high operating costs. If cementation of the fill is necessary, pneumatic systems often require a separate waterline to provide for mixing and hydration of cement. Other problems include larger pipe diameters needed for given fill rates as compared to hydraulic fill and higher energy costs. The systems are very noisy, with noise levels in the order of more than 100 dBA, or 85 dBA with commercial silencers (Reynolds 1989). Another potential *disadvantage* which must be considered is the possibility of sulfide dust explosions due to high sulfide backfill materials being conveyed pneumatically.

Pneumatic filling is not suitable for incremental mining with intricate workings. The circuit would require too many bends which would reduce the velocity of the conveyed material, making in-place densities lower. Much time and labour are required in preparation for pneumatic filling of a stope. The equipment used for pneumatic filling is large and difficult to move and install in small workings. As a result, a semi-permanent set-up is usually required. Finally, pneumatic fill does not produce a smooth working floor.

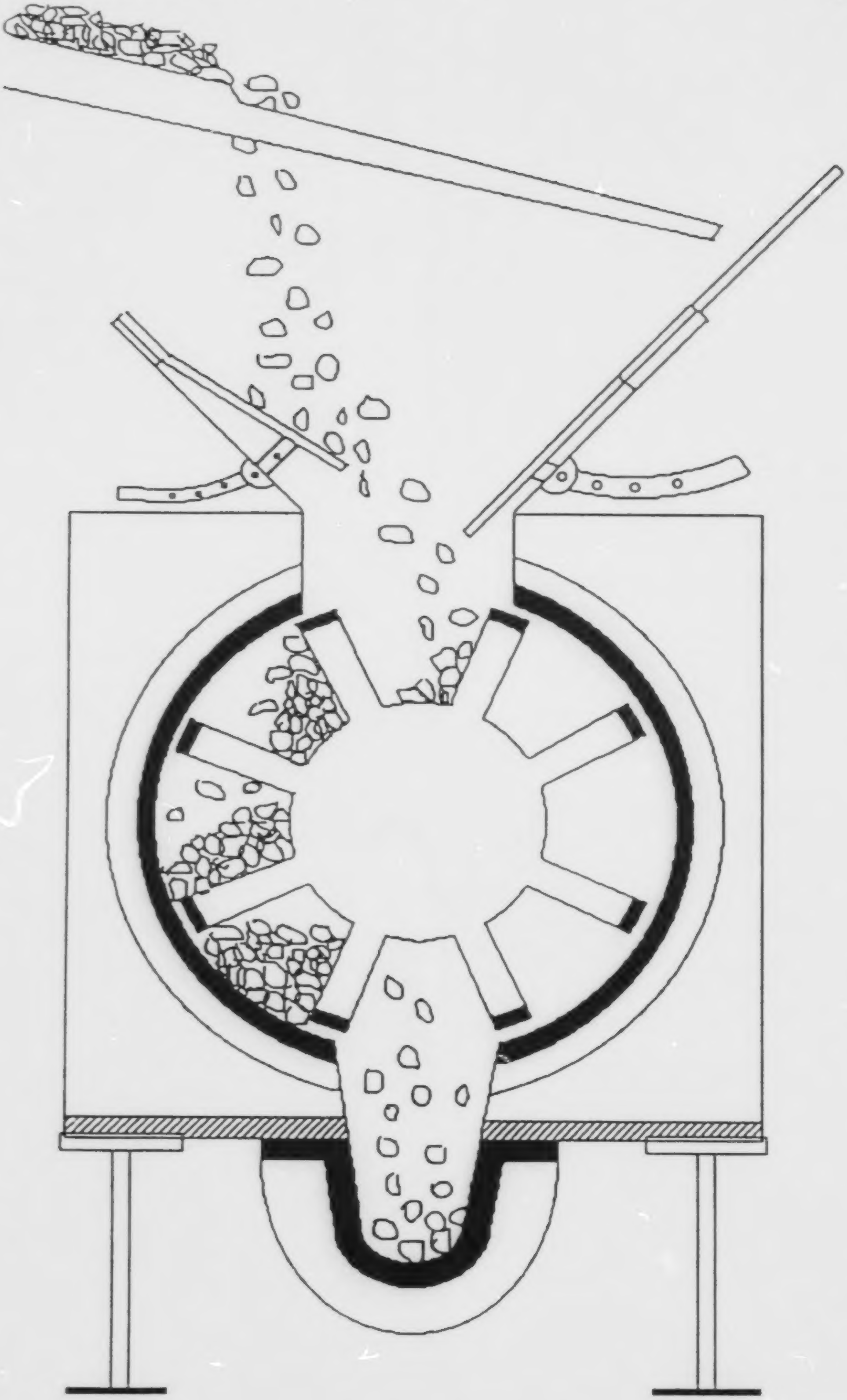


Figure 6.10, Schematic of rotating pocket feeder for pneumatic backfill, (After Reynolds, 1972)

6.4 FROZEN BACKFILL

Another minor backfilling method is frozen or ice filling. This method has found application in some operations, including one in Canada, but it is not widespread. This discussion on its theory and practice is included for completeness.

6.4.1 Background

The effect of freezing in consolidating unstable wet soils has long been known. Natural and artificial freezing has been employed to allow excavation to proceed in otherwise unstable materials. In mining, freezing has most commonly been employed to allow shaft sinking through wet overburden. Freezing also has its place in backfilling and several techniques for using ice or frozen sand or tailings mixtures have been proposed. Very few of these techniques have been tested in full-scale field trials and only one has been employed as standard practice in a Canadian mine.

The benefits of frozen fill are numerous, but the high strength developed as a result of freezing is of prime interest. As saturated ground is frozen, increases in compressive strength are attributed to water in the material acting as a binding agent for the solid particles. In turn, the particles provide inter particle friction and particle interlocking, thereby further increasing the strength of the frozen mass. The internal friction provided by solid particles in ice prevents plastic flow of the frozen mass when loads are applied.

Once all the water in a material has been frozen, strength can still be increased by further lowering the temperature of the mass. Also, if the frozen material fails due to excessive loads, the cold environment favours re-binding by restructuring of the ice bonds which cement the aggregate particles together. This "self-repairing" property is exclusive to frozen backfill.

Tests conducted on frozen backfill materials indicated that cylindrical samples consisting of a mixture of 72 percent tailings and 28 percent water by weight, frozen to a temperature of -25°C, achieved an unconfined compressive strength of 24.8 MPa(Prudhomme 1983). One very important property of frozen tailings which was observed during the testing was that even after failure, frozen tailings would still support a considerable amount of load which was estimated to be almost 50 percent of that of the intact sample. Similar studies by Archibald and Nantel (1983) indicated that a frozen tailings mixture containing 25 percent water and 75 percent tailings by weight achieved a uniaxial compressive strength which was almost double that of non-frozen, very strongly cemented backfill containing a 5:1 sand to cement ratio. Uncemented frozen tailings fill was also found to have a strength in excess of 15 times that of a conventional cemented tailings fill containing 30:1 sand to cement.

Pure ice also has support capabilities, but it attains a lower compressive strength than ice and solid particle mixtures because of the absence of internal friction components. As a result, pure ice is subject to a large degree of plastic flow or creep. Frozen backfill material containing solid particles will also be subject to a limited degree of creep. The properties and especially the strength characteristics of frozen backfill material depend on time and rate of strain.

6.4.2 Naturally Frozen Aggregate Fill

There is only one known operating mine which has relied on naturally frozen support in underground workings: Cominco's Polaris Mine located on Little Cornwallis Island in the Canadian Arctic. Here, the stability of all openings in the underground ore zone are dependent on the presence of the permafrost environment (Drake & Keohane 1985). Open stoping is practiced and openings are backfilled to permit later mining of the adjacent ore. The backfill must be competent enough to support a free-standing height of 40 meters and it must be of low enough temperature to prevent upsetting the natural thermal equilibrium in the mine. The ore has to remain frozen at all times. Shale outcrops on surface were quarried and mixed with ice or snow to produce a frozen rock fill which was dumped from surface through six foot diameter raise-bored holes. Once underground, the frozen rock fill was distributed to stopes using a small bulldozer.

Polaris mine has also investigated various schemes whereby mill tailings could be dewatered and naturally frozen in the arctic air on surface before placement underground as fill (Drake & Keohane 1985). The best known of these schemes is to pelletize the tailings on surface using sloped pelletizing disks prior to freezing, as described in a study by Archibald and Nantel (1983).

6.4.3 Artificially Frozen Aggregate Fill

6.4.3.1 Circulating brine solution

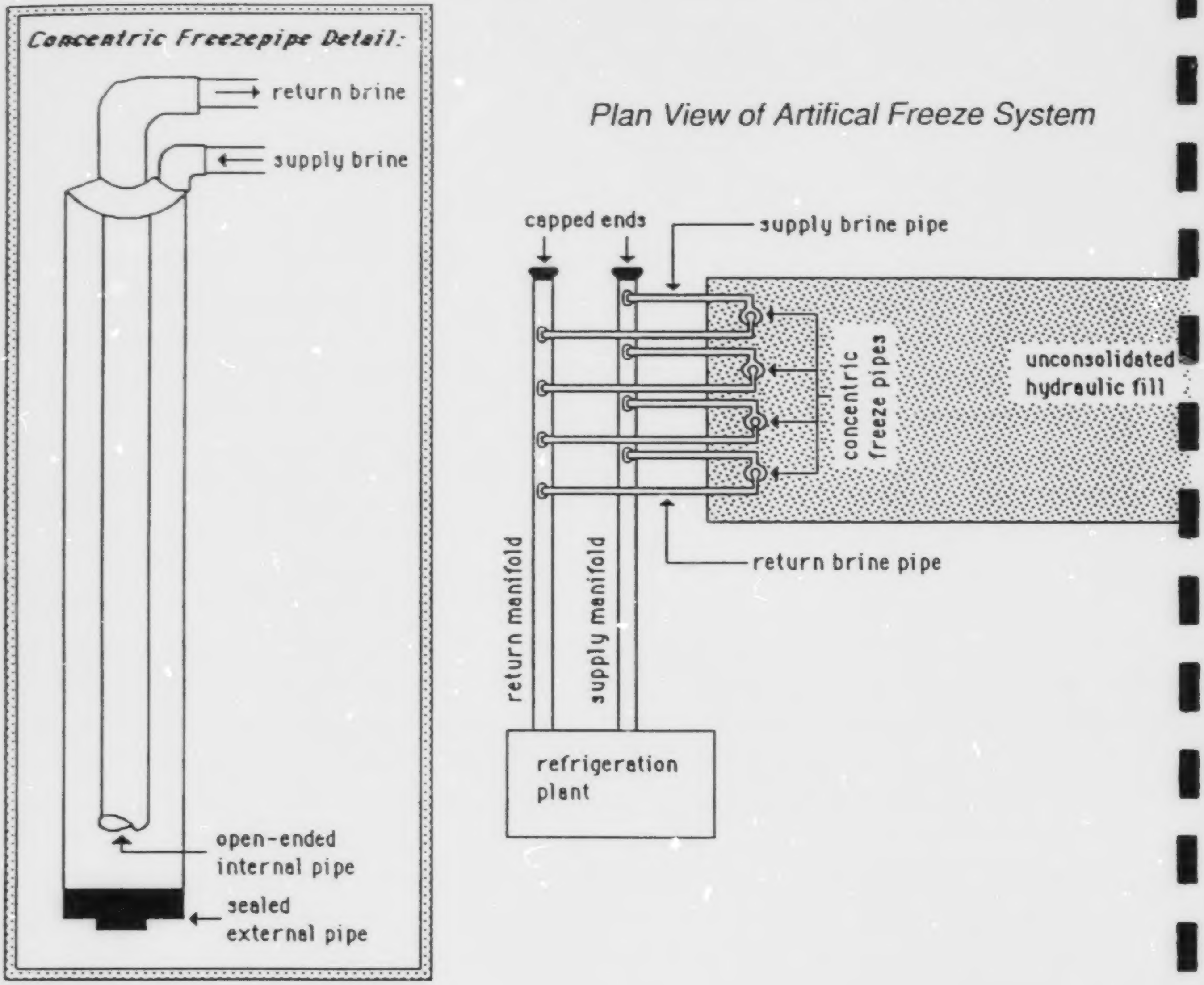
The use of a refrigeration plant and freeze pipe combination to freeze backfill in place, similar to a technique widely used to stabilize wet overburden for shaft sinking, has been employed in operating mines in Finland. Here, a curtain of backfill next to a pillar to be recovered is artificially frozen to provide a strong, stable retaining structure for the bulk of unconsolidated fill behind it.

The proposed method involves the installation of commercially available refrigeration units near the stope to be stabilized. The refrigeration unit produces a cold (-25°C) brine solution which is pumped through a system of freeze pipes placed directly in the backfill. Each freeze pipe consists of two concentric pipes placed one inside the other. The inner pipe is open ended and the outer pipe is sealed at its bottom so that refrigerated brine circulates down the outside pipe and back up the inside pipe. All freeze pipes are connected to large feed and return manifolds conveying brine to and from the refrigeration unit. Ideally, the freeze pipes would be installed in the stope prior to backfilling. Alternatively, holes could be drilled and cased in existing uncemented fill for post-consolidation, but this at a much higher cost. The freeze pipe concept and arrangement is shown in Figure 6.11.

Once the brine system is activated, a cylinder [illegible]zen ground of gradually increasing diameter begins to form around each pipe. The limits of the frozen cylinders continue to grow until overlapping occurs between adjacent cylinders. Continued expansion of the freeze limit eventually results in a wall of frozen backfill along the proposed mining face. When sufficient freezing has occurred, brine circulation is maintained at a reduced rate to prevent thawing or weakening of the wall. The concept is shown in Figure 6.12.

6.4.3.2 Cooling air system

An alternative method of producing an artificially frozen backfill without the use of circulating brine has been proposed by Mindelco of Sweden (Anon 1983). The technique is called "True Hydraulic Backfilling" (THB) and it was developed to provide a stable frozen roof in mines employing multiple pass undercutting methods for mining coal. For this technique, a stope to be filled is pre-cooled by circulating cold air (-15 to -25°C) through it. The air follows designated cooling-air loops to prevent over-cooling of other working areas. Once the stope and surrounding rock walls have reached an optimum temperature, thin layers of water and tailings are poured into the stope and are frozen again by the cold circulating air. During successive filling by thin layers, channels are left open to provide passageways for a small amount of cold air to be circulated for maintaining the ice after filling is complete. The air is cooled by refrigeration plants, unless natural air of sufficiently low temperature can be brought to the stopes from surface. Mining then proceeds under the frozen mass. A schematic diagram showing the method of mining using THB is given in Figure 6.13.



Concentric Freezepipe Detail:

Plan View of Artifical Freeze System

return brine
supply brine
open-ended internal pipe
sealed external pipe
capped ends
supply brine pipe
concentric freeze pipes
unconsolidated hydraulic fill
return brine pipe
return manifold
supply manifold
refrigeration plant

Figure 6.11, Placement of freeze pipes and manifolds for artificial freezing of backfill, (After Prudhomme, 1983)

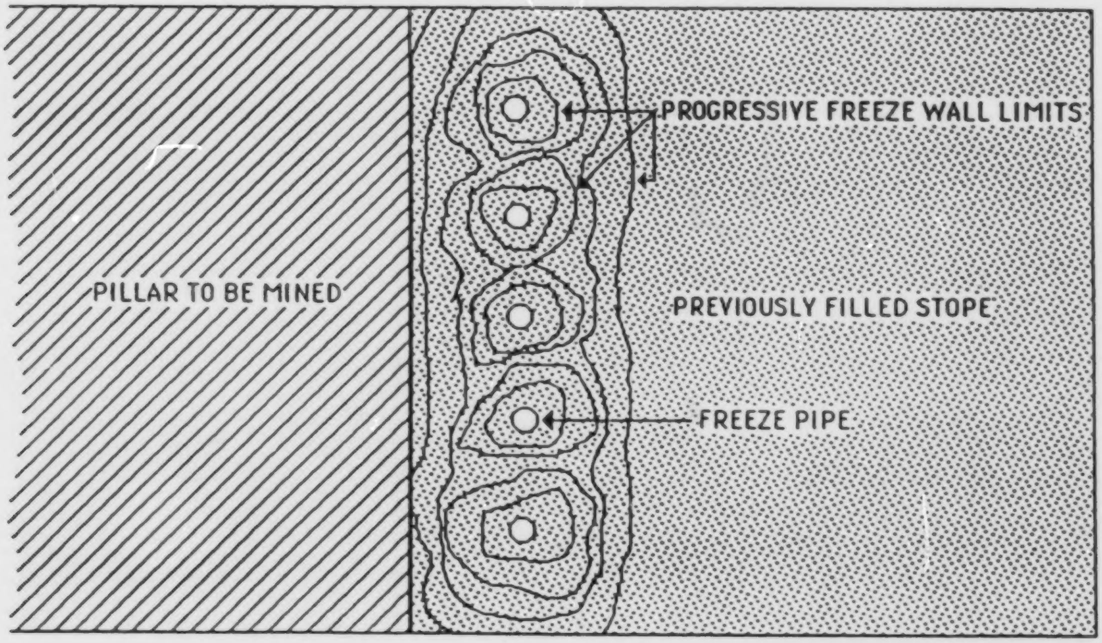

PROGRESSIVE FREEZE WALL LIMITS
PILLAR TO BE MINED
PREVIOUSLY FILLED STOPE
FREEZE PIPE

Figure 6.12, Plan of freeze pipes and progressive growth of frozen fill wall, (After Prudhomme, 1983)

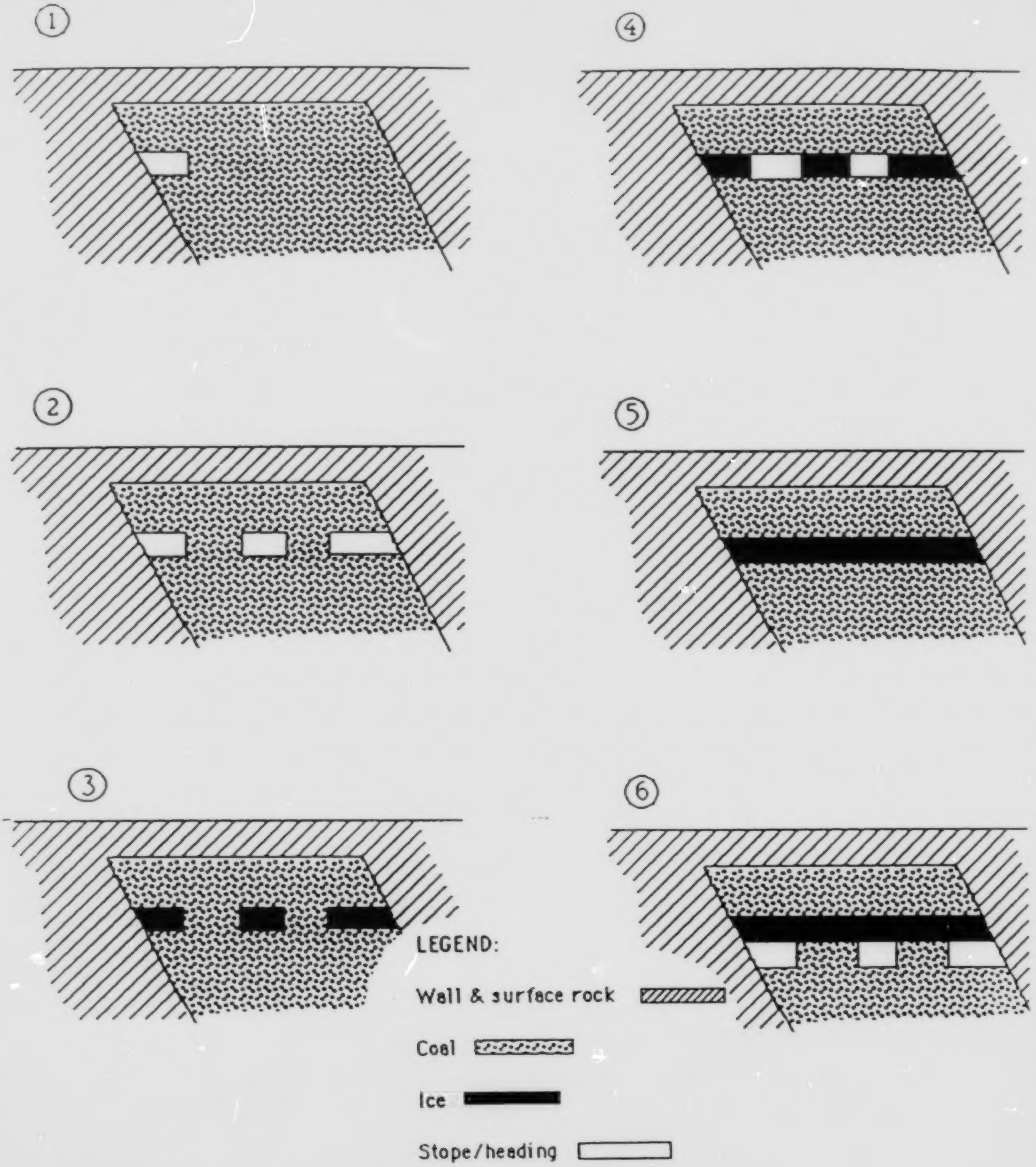

①
④
②
⑤
③
⑥
LEGEND:
Wall & surface rock ▨
Coal
Ice ▬
Stope/heading ▭

Figure 6.13, Section view of undercut mining employing the THB method of ice filling. (After Anon, 1983)

6.5 CEMENTITIOUS FOAM

6.5.1 Background

Cementitious foam, first introduced in the mid-1980's, has been used for filling voids in underground coal mines in the U.K. This foam has a very light consistency and is similar to shaving cream in texture (Anon 1989a). In the late 1980's, pumped foam was employed with fine and coarse aggregates to produce foamed concrete mixes for surface construction purposes, (Anon 1988). The lightweight foamed concrete, having densities between 350 and 1800 kg/m^3, was produced for concrete pumping operations with output rates of up to 60 m^3 per hour.

Success with the above operations prompted one company to market a cementitious foam product to the Canadian hard rock mining industry. The product, marketed as "Tekfoam", was first introduced to hard rock mines in Canada in June 1988 (Technik Mining, undated). Since then, other Canadian companies have also supplied cementitious foam suitable for mining uses.

The Canadian applications for cementitious foam products range from general void-filling to the construction of fill fences to contain large backfill pours for VRM and blasthole stopes. Cementitious foam has also been used to construct ventilation seals and stoppings. From its introduction in June 1988 until July 1989, approximately 500 tons of the Tekfoam type had been used for a variety of Canadian hard rock applications, such as (Technik Mining, undated):

1. Foam as a cavity filler:
   - for the stabilization of old stoping boxholes to permit pillar recovery.
   - for rapid stabilization of spalling ground in a square set top sill close to a major fault zone.
   - for the stabilization of ground in an abandoned ramp area where a major fault zone was encountered.
   - for the stabilization of old drift walls where placed sandfill had sloughed, resulting in a progressive cavity situation.

2. Construction of fill fences using foam:
   - as a fill fence for a large blasthole stope.
   - as a barricade for low back areas where backfill is required to be tight to the back.
   - as perimeter sealing material for conventional timber fences.

3. Foam for ventilation seals:
   -as a temporary ventilation barrier where frequent blasting damages occur to conventional seals.

There are two specific Canadian applications of foam that are well documented in the public domain literature (Clarke & Haaranen 1989, Anon 1989). These report field trials at INCO Ltd. in Sudbury, Canada, where foam was used to construct a fill barricade for a VRM stope at the Creighton Mine and to stabilize old blasthole stoping boxholes at the Levack Mine. The Creighton foam fill barricade measured 6.6m wide by 3.7m high by 4.5m thick. Pumping time for the structure was nine hours. A containment fence consisting of filter cloth hung from screen with four 51 mm braces was erected. The outside containment was a single four by eight sheet of plywood held as a temporary form until the foam was set, which took 60 to 90 seconds. The wall had a designed compressive strength of 240 KPa. The only problem encountered was due to perforated drain pipes placed through the barricade, which caused the foam to dry-out in a chimney effect at the perforations. This left a void around the diameter of the pipe. Drying of the foam on the barricade's outside face extended inward about 25 to 50 mm, but did not pose a structural threat.

The Levack Mine boxhole filling procedure using foam was deemed successful because the old boxhole timbers would not have been strong enough to support concrete or cemented sandfill. The cementitious foam was lighter and placed less demands on the supporting timbers. The rapid setting time allowed the foam to become self-supporting in less time than concrete or conventional fills.

6.5.2 Benefits

Tekfoam is described by Technik Mining as a non-combustible cavity filling foam which is safer to handle than cement. Technik claims that the product has many advantages. It produces no harmful vapors and very little dust. The company also boasts that the product is capable of high yields and fast setting times. With this system, it is claimed that the operator can be positioned safely even in remote and hazardous areas. Tekfoam is said to be practical to use in confined areas or where ground is too badly broken for normal backfill types. This is because Tekfoam is very easy to support, as only screen and filter cloth with some minor bracing are sufficient to hold it in place. No elaborate barricades or fill fences are required. Also, openings in the foam of up to 50mm in diameter are self-sealing in very little time. An added bonus is that there is no water or slime runoff, eliminating the need for drainage facilities and clean-up activities on filling levels.

6.5.3 Foam Production and Foaming Equipment

Foam for backfilling is produced by creating bubbles within a cementitious slurry mix. The bubbles are formed by physically mixing air with the slurry. They are not formed by the chemical generation of gas within the mix. Once bubbles are formed, they are stabilized by foaming agents in the mix until the cement portion hardens and sets the foam. Tekfoam comes in a powder which is supplied in 25 kg poly-lined bags.

A special foam placer machine consisting of a continuous mixer and pump is needed to prepare the foam. A schematic of the foam placer machine is shown in Figure 6.14. Air and pre-determined quantities of powder and water are mixed at the foam placer and are then passed through a discharge hose. The amount of air entering the foam placer is not under the operator's direct physical control because the pump operates at a fixed output volume speed. Therefore, the amount of air is governed by the volume of slurry supplied to the pump. The greater the volume of slurry supplied, the less air will be mixed. The water to solids ratio is usually varied between 1.25 to 2.5, and the air content ranges between 78 and 87 percent by volume.

Technik recommends a one inch diameter flexible discharge hose of 50 to 60 meters length for optimum placement. A short sleeve of 50 mm diameter hose should be appended to the discharge end of the main hose to slow the jet output. Under normal operating conditions, foam can be pumped to a distance of a few hundred meters.

6.5.4 Properties of Foam Fill

The relative amounts of powder and water metered into the machine affect the properties of the foam. Higher strength of the solidified foam is achieved by reducing the water to solids ratio and the air content. Hence, in order to obtain a stronger mix, it is necessary to reduce the amount of water and to increase the supply of cement slurry to the foam placer. This also results in a slightly more rapid setting time and a larger yield.

Compressive strengths of typical foam mixes vary between 0.11 and 0.33 MPa (15 to 50 PSI) with a setting time of 6 to 18 minutes at 20°C. When a higher solids content (i.e. higher strength mix) is required, 24 hours of setting time is recommended to achieve ultimate strength. If long-term integrity is required in the solidified mass, Technik advises that it is necessary to prevent the surface exposures of the foam from drying out. Tekfoam must remain in a humid condition to be effective. A typical example of what can happen if the foam dries out was described in section 6.5.1 where drying occurred around a perforated drainpipe put through a foam fill fence poured at INCO's Creighton Mine.

Factors which affect the performance of the foam after placement are water content, temperature during placement and curing, pumping rate, discharge hose capacity and mass volume of product placed.

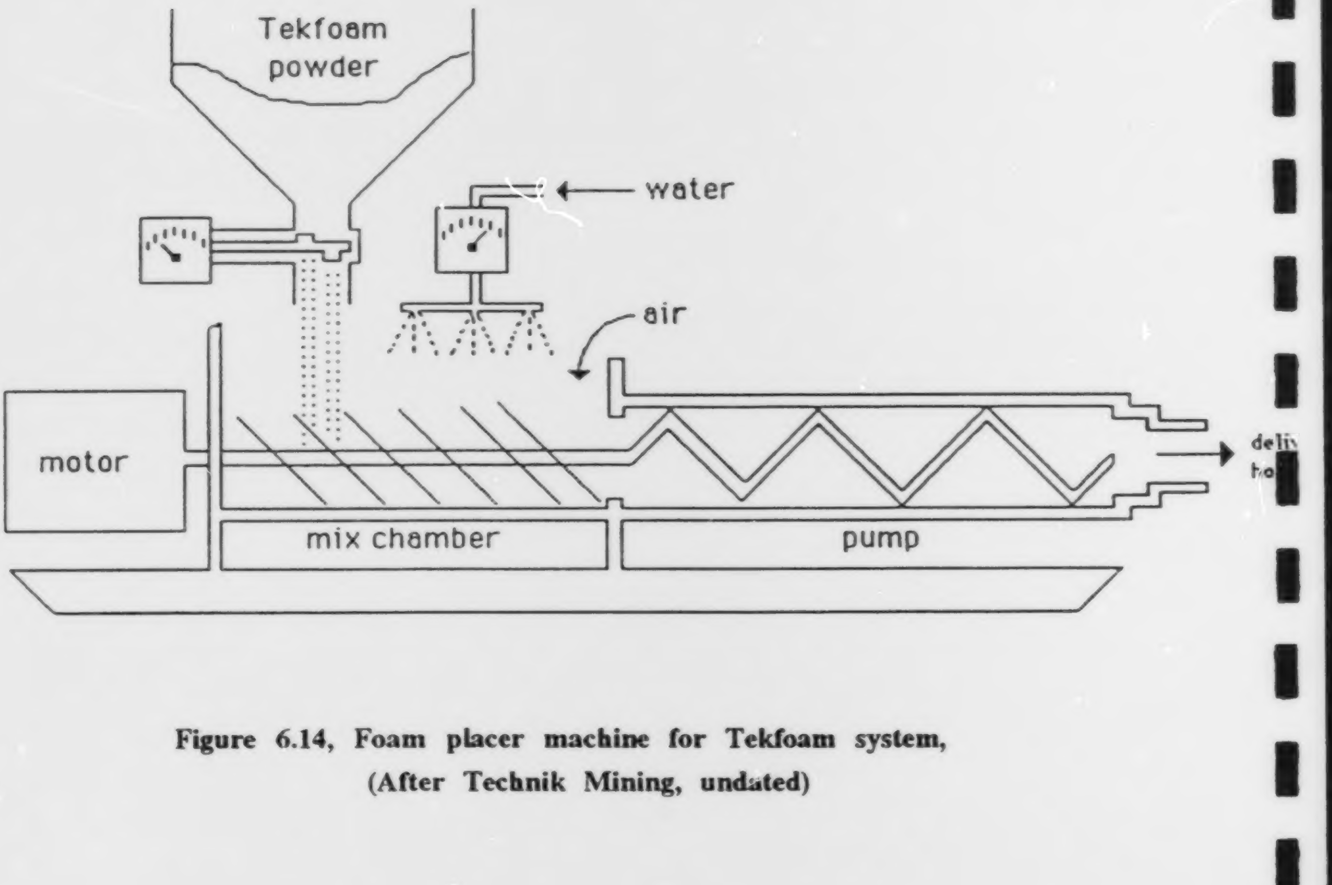

Tekfoam
powder
water
air
motor
mix chamber
pump
deli
ho

Figure 6.14, Foam placer machine for Tekfoam system, (After Technik Mining, undated)

6.6 MONOLITHIC PACKING

6.6.1 Background

First introduced in 1973, monolithic packing is a technique widely employed in coal and other soft rock mines to prevent subsidence and reduce overall deformation in workings. In brief, monolithic packing consists of a two part cement with each part being prepared, slurried and pumped separately. When the two parts are combined, they set very rapidly (within minutes) to form a strong cement product. It is used to create artificial pillars by pumping into light weight form work. Forms into which the slurries are pumped consist of water-tight fabric bags supported by wire mesh and timber bracing. Several commercial pump packing systems are available under various trade names such as Aquapak, Tekpak, Astrapak, Anpak, Flashpak, Thyssen packing system, Tekblend, etc.

Monolithic packing materials consist of a mixture of high alumina cement, bentonite, anhydrous calcium sulfate (anhydrite) or flyash which, when mixed with water and an accelerator such as sodium carbonate, form a quick setting slurry. Retarders can also be added to the slurries to control the setting time.This is especially useful when long pumping distances are required between the mixing station and the packing site.

There are many positive features associated with the use of such packs. For example, they are non-combustible, non explosive and they do not produce noxious gases. They are water-resistant and air-tight. They are simple and rapid to prepare and place and can, if desired, incorporate some mine waste. They cure and set very rapidly and hence they develop a rapid early strength and they can be self-supporting within 15 minutes. Their ultimate strength is reached in four to seven days, depending on the pack materials used. Packs are of variable width and height and are pumped to conform to the geometry of the installation site, thereby always offering a tight fit. Shrinkage effects are very low and therefore, packs remain tight even after a considerable amount of time. And finally, they have good residual strength, meaning that even if failure of the pack occurs, it can still maintain a considerable load.

The use of monolithic packs is of interest to the hard rock mining industry for potential application as tight filling material, where the pack slurry could be pumped to fill the gap between the top of the existing fill and the back. It could also be used as a filler or plug in conjunction with fill fences or alternatively as a construction material for the fence. Another potential application would be for spot filling of localized problem areas, such as cavities left from sloughed stope walls. The cost of the product is generally such as to preclude their use as a mass filling system.

6.6.2 An Investigation into Design of Tight Packs in Hard Rock Mine Backfill

As part of this project the Department of Mining and Metallurgical Engineering, McGill University, Montreal Quebec was commissioned to carry out a study into the potential for use of monolithic packing materials in hard rock mine backfill applications. The study was primarily conducted to evaluate the potential of anhydrite as a binder when mixed with mine tailings for underground backfill. In the process a through review of monolithic packing products was conducted. The full report is included as appendix J. A summary of this review is presented below in table 6.1.

COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS

(AS OF 1985 PRICES)

Pack Type	Material Costs (CAN$/$m^3$ of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Aquapak	Aquabent	50	12.50		0.52	1.30	4.00	5.00
	Aquacem	450	95.86	108.36				
Thyssen	Coal	588		0.45	0.48	0.95	1.34	
	Plowmat	16	2.16					
	Packbind	298	64.44	100.17				
Warbrat	Coal	588	33.57		0.07	2.40	3.10	4.90
	Plowmat	16	2.16					
	O.P.C. (Bulk)	298	23.53					
	TEA 51	30	24.03	83.29				
Takpak	Takbent	182	40.85		0.67	2.50	4.0	4.2
	Tekcem	182	57.95	98.80				
Astrapak	Astrabent	183	37.75		0.5	1.4	4.4	4.5
	Astracem	183	52.96	90.71				

Table 6.1

COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MANOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/m^3 of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Wieght (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Flashpak	Fly Ash	921	27.63		0.02	0.9	4.1	9.3
	O.p.c. (Bulk)	255	41.31					
	$FeSO_4$	100	22.50					
	Fulbent	18	5.94	97.38				
	Fly Ash	930	30.63		0.40	1.6	4.4	8.2
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	65	24.62					
	Pulbent	95	31.67	128.23				
	Fly Ash	930	30.63					
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	43	16.29	88.23				
Preblend	Limestone$^+$	2000	73.59		0.78	1.53	8.80	9.80
	Rapid Setting Cement (Bulk)			73.59				

Table 6.1 Con't

COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/$m^3$ of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Anpak	Synthetic Anhydrite (Bulk)	2000	39.79		0.26	6.1	21.0	28.0
	Accelerator	20	8.65	48.44				
	Natural Anhydrite (Bulk)	2000	19.90		0.05	0.8	14.5	20.3
	Accelerator	56	24.22	44.12				

Table 6.1 Con't

6.6.3 Types of Monolithic Packs

The following truly monolithic packing materials were reviewed.

1. Aquapak. A specially developed rapid setting cement (Aquacem) in slurry form is mixed with a bentonite and accelerator slurry just as it is pumped into a brattice bag where it rapidly sets.

2. Aqualight. A light weight foamed cementitious material.

3. Astrapak and Tekpak. Similar products based on rapid setting cement which is mixed with a bentonite and accelerator just prior to placement.

4. Anhydrite pack (Anpak). Anhydrite ($CaSO_4$) will in the presence of water, form dihydrite ($CaSO_4.H_2O$) a strong solid material. The reaction is relatively slow so that an accelerator is introduced; potassium sulfate and hydrated iron sulfate.

5. Flashpak. A portland cement and flyash mixture with accelerator and bentonite. A cement slurry and a flyash, bentonite and accelerator slurry are prepared separately and pumped to the placement location. Mixing occurs in the spray nozzle and it gels almost immediately.

A pump pack is a continuous concrete packing system using monolithic materials similar to those described above but including aggregate for better strength. A typical system will use run of mine coal or washery rejects as aggregate. The aggregate is slurried with bentonite to act as a flow suspendent, it is pumped to the face and mixed with a cement and accelerator slurry just prior to placement. A number of manufacturers market pump packing system.

1. The Thyssen system.

2. Warbret system. Used by NCB in Britain, a modification of the Thyssen system.

3. Short range systems. Batch systems not common in coal mines.

More detailed descriptions of these materials can be found in appendix J.

6.6.4 Trials with Monolithic Packing for Fill Fence Construction

Field trials involving monolithic packing to produce a fill fence were conducted by Falconbridge Limited, Sudbury Operations at the Lockerby Mine. The Tekpak system was employed. The fence was constructed of three rows of two pack bags filled with Tekpak staggered in brick wall fashion. The fence as constructed was 5.2m wide by 4.9m high and 1.2m thick. The fence construction was a modified cable fill fence design with some timber included for additional support of the pack bags. Figure 6.15 shows the mixing arrangement used and figure 6.16 the fence construction.

The trial was a success, the monolithic fence had no problems holding the pressure of hydraulic fill placed behind it. However the practical and economic aspects of using monolithic packing on such small batch jobs were not favorable. The technique was labour intensive unless done on a large scale with specialized equipment. The material costs were $189 per cubic metre (1988 Canadian dollars) which is one and a half times the cost of type 10 portland cement. In practice most problems occurred with handling the pack bags, ensuring that they were not ripped, and that they filled correctly. The current bag technology was not up to the application.

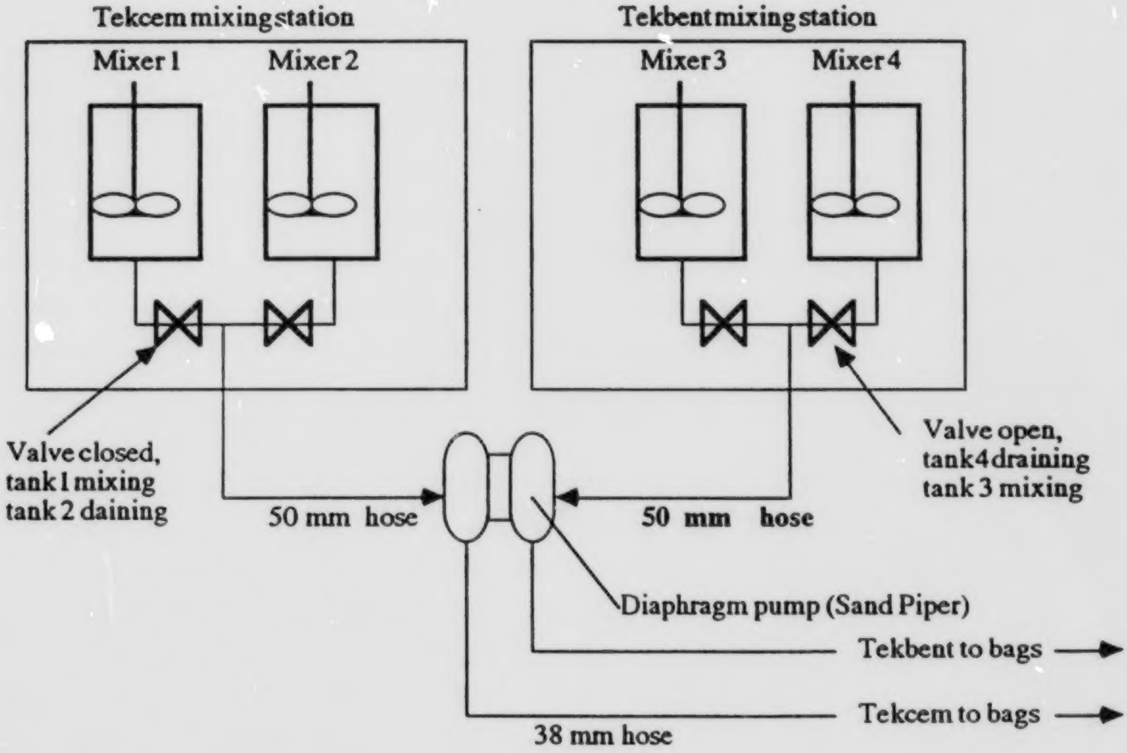
Monolithic packing systems are interesting and can be used to form structures underground. However, at the present level of development they are handicapped by being suitable for operations where structural material is consumed at a rate of hundreds of cubic metres per week while this application would consume hundreds of cubic metres per year. The system requires careful attention to detail for the best results, so it does not lend itself to occasional use by inexperienced personnel.

When selecting a pump packing product for a mining application, there are many factors in addition to basic cost which should be considered. For example, a suitable product requiring the least amount of raw materials should be chosen so as to minimize transport requirements. The setting time of the pack mix should be carefully selected based on pumping distance and early strength requirements. For very long pumping distances, the product's resistance to scaling or blocking in pipes is important. Distances of up to 5.5 km have been successfully achieved with specific products designed for such applications (e.g. Tekpak XX WRH/1). The product should offer a shelf life which is compatible with anticipated pack scheduling. The aggressiveness of the chemicals involved should be known so that precautions can be taken if it contains any skin irritants. Finally, the product's sensitivity to site-specific factors such as acidity of mine water should be assessed to determine any detrimental effects.

6.6.5 Other Uses for Monolithic Packing

Rockfill capping has been proposed as a use of monolithic packing in hard rock mines. Here the monolithic material would be used to form an impermeable layer on the top of the rockfill over which hydraulic fill could be placed to give efficient tight filling of a stope. This application was investigated by Kidd Creek Mines as part of this project, and the results are discussed in Chapter 5.

Figure 6.17 shows other conceptual designs for which monolithic packing materials might be suitable.


Tekcem mixing station
Mixer 1
Mixer 2
Tekbent mixing station
Mixer 3
Mixer 4
Valve closed,
tank 1 mixing
tank 2 daining
Valve open,
tank 4 draining
tank 3 mixing
50 mm hose
50 mm hose
Diaphragm pump (Sand Piper)
Tekbent to bags
Tekcem to bags
38 mm hose

Valves set so that one tank in each station is mixing while the other is being drained, this will provide proper batch mixing but with continuous supply to the pump. Each side of the pump handles only one product, mixing takes place in the bag.

Figure 6.15, Monolithic packing bag mixing arrangements.

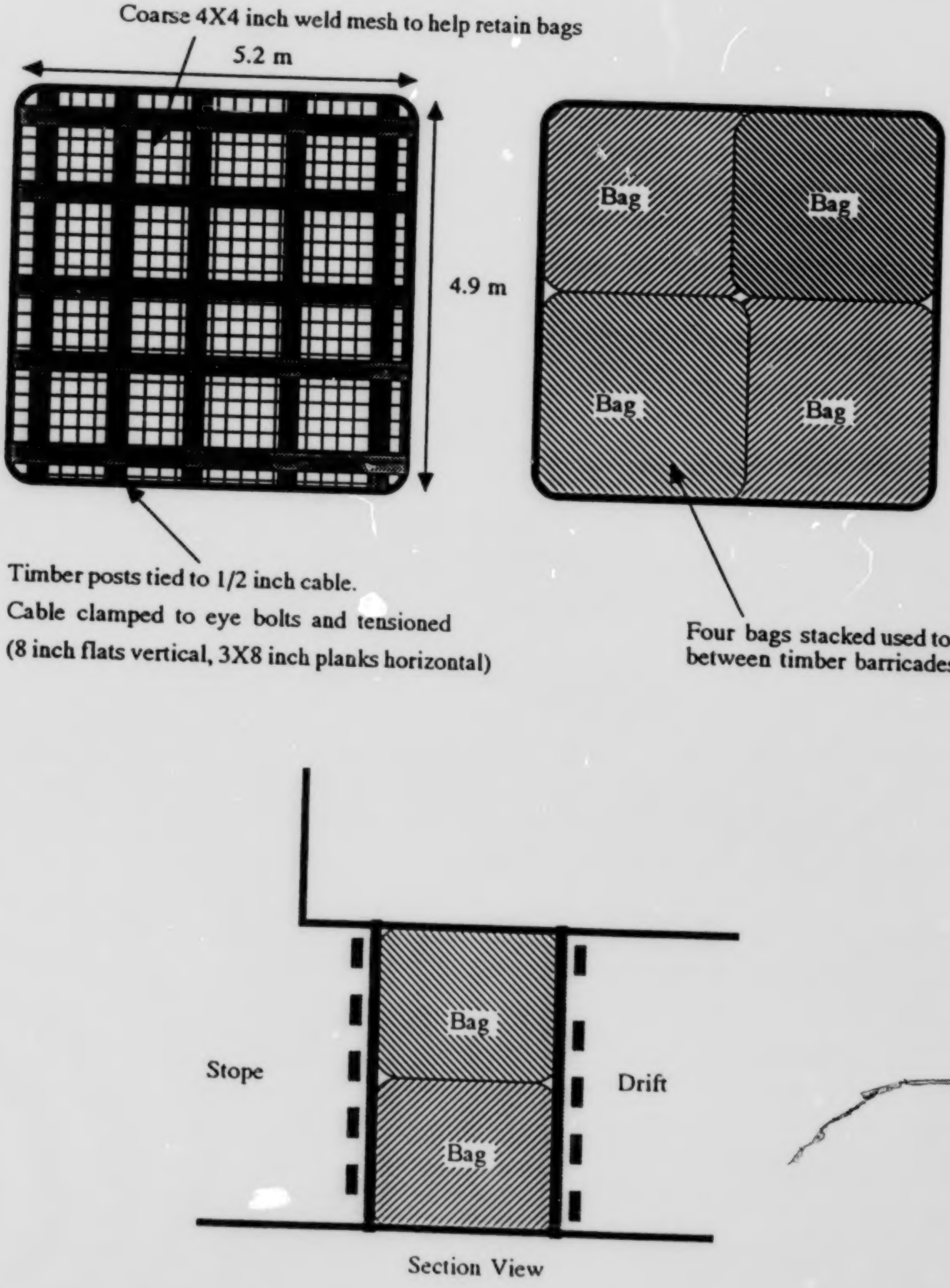

Coarse 4X4 inch weld mesh to help retain bags
5.2 m
4.9 m
Timber posts tied to 1/2 inch cable.
Cable clamped to eye bolts and tensioned
(8 inch flats vertical, 3X8 inch planks horizontal)
Bag
Bag
Bag
Bag
Four bags stacked used to fill between timber barricades
Stope
Bag
Bag
Drift
Section View

Figure 6.16, Monolithic packing fill barricade design.

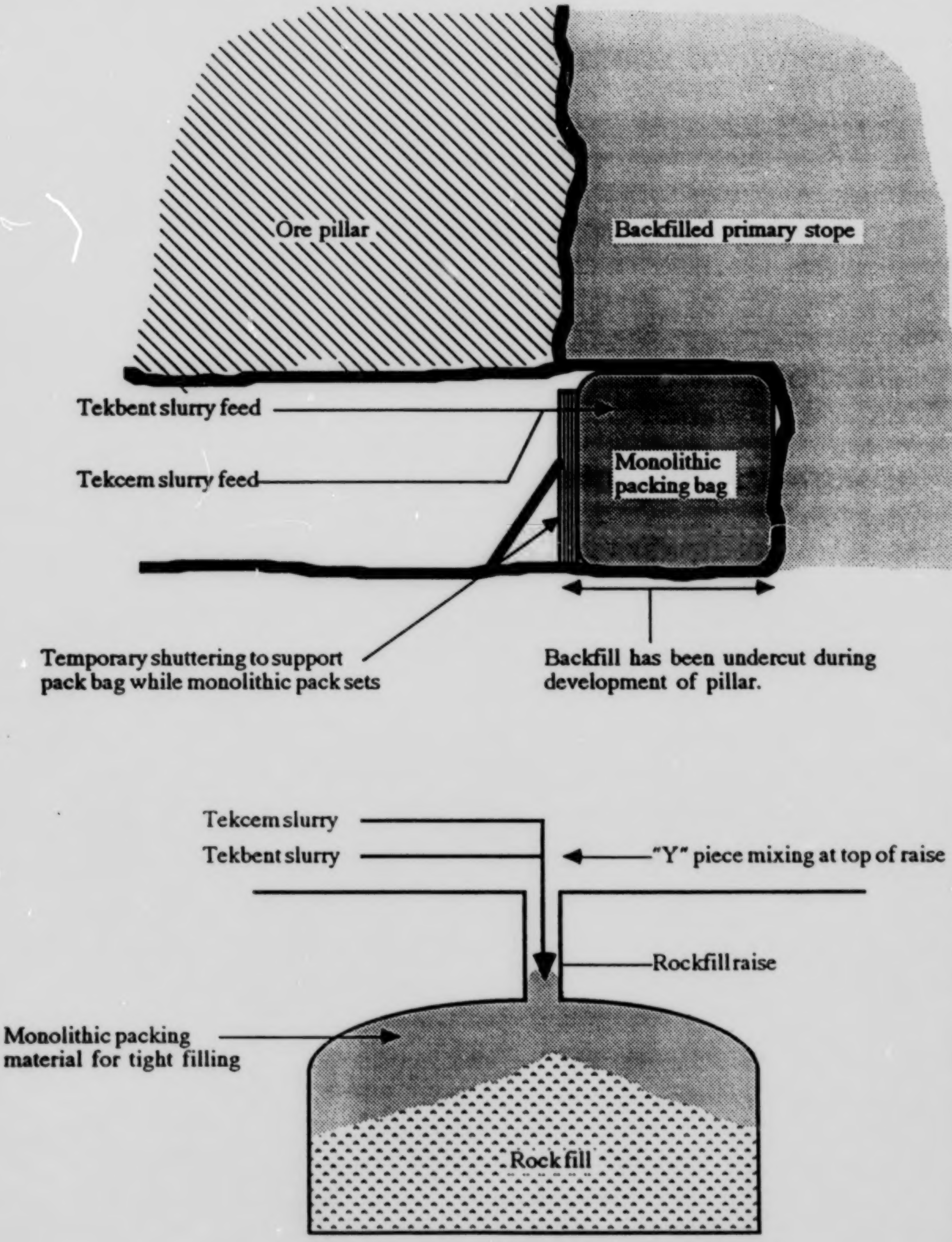

Ore pillar
Backfilled primary stope
Tekbent slurry feed
Tekcem slurry feed
Monolithic packing bag
Temporary shuttering to support pack bag while monolithic pack sets
Backfill has been undercut during development of pillar.
Tekcem slurry
Tekbent slurry
"Y" piece mixing at top of raise
Rockfill raise
Monolithic packing material for tight filling
Rock fill

Figure 6.17, Possible applications for monolithic packing materials.

6.7 REINFORCING ELEMENTS IN HYDRAULIC FILL

6.7.1 Background

Reinforcing elements have been successful in improving strength, impact resistance and shrinkage behavior in concrete and shotcrete. There is potential for them to do the same thing for consolidated hydraulic fills. The reinforcing elements being considered here are fibres, mesh elements, geotextile screens. There has been sufficient research work conducted to be confident that reinforcing elements do improve the strength of consolidated fill. The main concern with their use is, do they work well enough to make them an economic backfill alternative. A series of centrifugal model tests were conducted as part of this project to investigate the practical use of reinforcing elements.

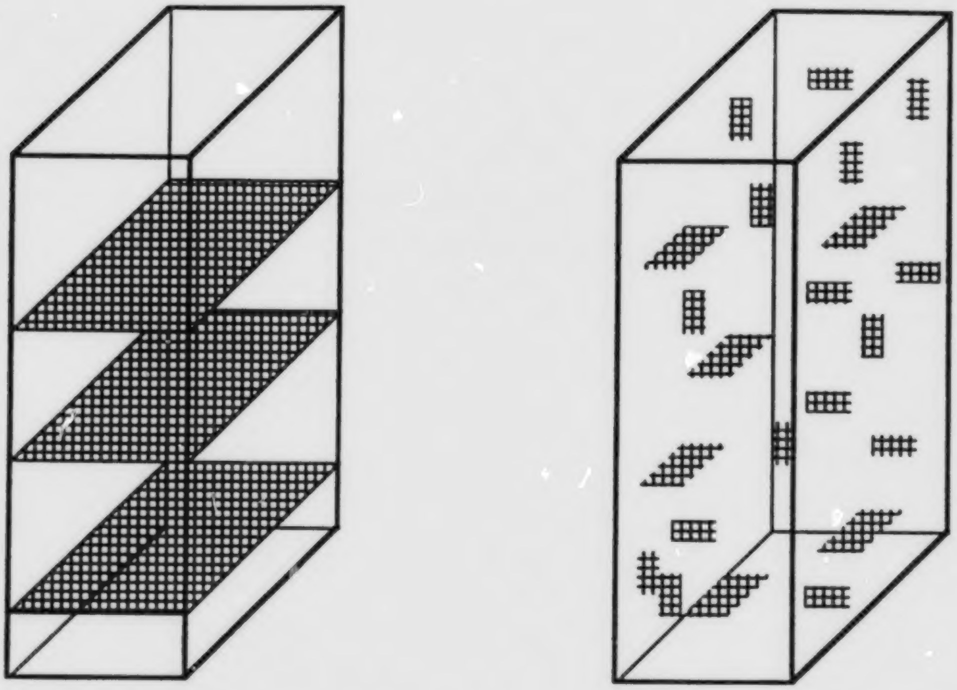
Reinforcing fibres can be steel or synthetic in composition and can have various configurations. Steel fibres can be plain linear fibres of varying lengths, or they can have kinked or crimped ends or middles. Others have additional steel to form anchors placed perpendicularly at each end to form an I-shape. Such modifications at the ends of fibres permit better anchoring of the fibre in the hardened backfill material.

Synthetic fibres, usually made of polypropylene, can be fibrillated to produce an anchoring effect similar to that of crimped steel fibres. Synthetic fibres may be preferable in mining applications where highly corrosive environments may not allow steel fibres to have a sufficient service life. On the other hand, the chemical and combustion characteristics of synthetic elements should conform to mining requirements.

Mesh elements are available in various screen hole size and sheet dimensions. Products commonly known as "geogrids", "snippets" and "fibermesh" are included in this category. Mesh reinforcing elements such as snippets are usually employed in 50 mm by 100 mm sheets composed of a five by ten grid of 0.5 mm thick plastic strands.

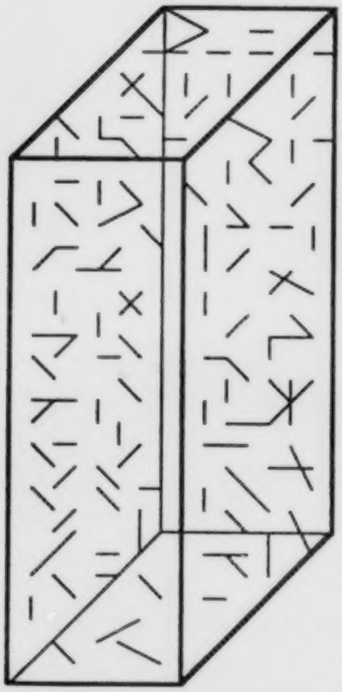
Geotextile screens found their original application in stabilizing structures such as embankments in civil engineering projects. They are simply blankets of geotextile material placed as an integral strong element in the fill.

Reinforcing elements can be added to the hydraulic backfill before pouring so that they are transported simultaneously with the bulk fill. Sometimes this can cause a problem, especially where larger mesh elements are concerned, because it can lead to plugging and blockage in distribution systems. Alternatively, the elements can be placed separately, but this requires access to stopes during the filling operations.



Geo grid reinforcement

Mesh element reinforcement



Fibre reinforcement.

Figure 6.18, Hydraulic backfill reinforcement concepts.

6.7.2 Reinforcing Mechanism

There are various mechanisms involved in the reinforcing of backfill structures by elements. The mechanisms by which fibres increase strength are thought to depend on three factors: a) frictional resistance developed parallel to the fibres, b) mechanical anchoring at the kinked ends of fibres, and c) bending stiffness of the material from which the fibre is made. During failure of fibre reinforced cemented tailings samples, steel fibres, even those with crimped or kinked ends, were found to pull out of the matrix rather than break, indicating that the tensile strength of the steel fibres is never fully mobilized (Wittreich 1988).

Mesh-like synthetic polypropylene fibres provide better anchoring by design, but due to their low tensile strength, they tend to fail as the specimen fails. Mesh elements act by producing an interlocking between the backfill particles and the mesh. Shearing resistance can be mobilized by very little relative movement between the backfill and the mesh (Mercer et al 1984). Mesh elements also improve the existing cohesion of a material or alternatively, give a granular unconsolidated material some intrinsic tensile strength which has been termed pseudo or mechanical cohesion (Netlon 1986).

Tests conducted by Li and Mitchell (1988) indicated that various types of reinforcing elements increased the ductility of weakly cemented sand, allowing the material to absorb strains at least four times higher than unreinforced material. In backfill, this increased ductility could result in a fill which would be resistant to static and shock loading (Netlon 1986). Such fill would be less subject to damage as a result of blast vibrations, meaning less dilution of the ore with fill.

The effect of alignment of various reinforcing elements on strength of weakly cemented sand was also studied by Li and Mitchell (1988). It was found that different horizontal alignments of deformed synthetic and anchored steel fibres did not affect strength, but they affected ductility of the samples. The most advantageous alignment consisted of a horizontal arrangement with long fibres oriented perpendicular to the free face. Random patterns of the same type of reinforcement resulted in lower sample strengths. Li and Mitchell explain that the random fibres interfere with the uniform development of shearing resistance by causing stress concentrations or planes of weakness.

A full range of design equations for producing reinforced structures containing geogrids is available in manufacturer publications (Tensar 1986). These are not specifically applicable to backfill.

6.7.3 Centrifugal Model Tests on Reinforced Consolidated Backfill

As part of this project a set of laboratory modelling studies on the inclusion of reinforcing elements in cemented classified tailings backfill were conducted by Kingston Georesearch Limited as a subcontractor to Falconbridge Limited. The study conducted in 1988 was based upon centrifuge model testing, (see section on physical modelling for a description of centrifuge modelling).

The reinforcing elements studied were geotextile grids and mesh elements (snippets). Weakly cemented hydraulic backfill at cement tailings ratios of 50:1 and 33:1 was used in the testing of these elements. The height of stable fill exposure was used as a measure of successful reinforcement. Ordinary portland cement was used as the binder.

Results indicated that both types of reinforcement produced significant improvements in fill stability. Geogrids produced up to a seven fold increase in the stable free standing height of the weakly cemented backfill. The mesh elements were less successful, but produced up to a three fold increase in free standing height.

The mesh elements can be mixed and transported hydraulically in the fill pour. This allows for inexpensive placement. However there are some problems; the mesh elements tend to agglomerate in the pour and may become aligned in directions where they contribute very little to fill stability, the mesh elements can interact in the fill line leading to blockages.

The economics of mesh element reinforcement appear to be unfavorable at the present time. The cost of adding mesh elements to the backfill is higher than the cost of additional cement to achieve the same improvements in stable fill heights. This is of course site specific and individual operators should review their own costs. The introduction of lower cost binders such as slag and flyash will not help the competitiveness of mesh elements.

The placement of horizontal geogrids is more complex than for mesh elements. Safe stope access is required. A typical application might be primary cut and fill stoping where stope access is not a problem. The test results indicate the following grid spacings would be appropriate; for 50:1 backfill a 7m vertical spacing, 30:1 backfill a 10m spacing, uncemented fill a 3 to 5m spacing. The placement of geogrid reinforcement would not be practical for large blasthole stopes.

The economics of grid reinforcement are not clear because it would be very site specific, dependent on the labour and capital cost involved in their placement. It is conceivable that an economic system could be worked out.

Table 6.2 below gives the results of the modelling. Critical height is the height at which an exposed wall of fill failed under the action of gravity. Dynamic blast damage effects were not considered in this test work.

Tailing:Cement ratio	Reinforcement	Corrected Prototype Critical Height , m	Improvement Factor
50:1	3 layers of geogrid	23	3.3
33:1	"	36	2.7
50:1	5 layers of geogrid	31	4.4
33:1	"	65	5.0
50:1	9 layers of geogrid	52	7.4
33:1	"	78	6.0
50:1	1 Kg/m^3 of snippets	11	1.5
33:1	"	20	1.5
50:1	2 Kg/m^3 of snippets	15	2.1
33:1	"	26	2.0
50:1	3 Kg/m^3 of snippets	23	3.3
33:1	"	42	3.2

Table 6.2, Improvement in critical height of model fill pours due to reinforcement.

6.7.4 Reinforcing Fibres

The Lucky Friday Mine in the Coeur d'Alene district of Idaho U.S.A. has been the site of extensive testing involving the use of fibres in backfill. Wittreich (1988) reports on field trials where backfill roofs in undercut and fill mining were reinforced by the addition of steel and synthetic fibres. Steel fibres had crimped ends while polypropylene fibres were fibrillated to enhance anchoring. Fibre addition was varied from zero to 59 kg of fibres per cubic meter of fill having a 17:1 sand to cement ratio.

Results of the field trials indicated that there is an optimum steel fibre content for maximum improvement in compressive strength, which was found to be 47 kg per cubic meter of fill. Any further addition of fibres in excess of the critical value resulted in decreased strength. This was thought to be due to the high concentration of fibres and resulting low concentration of cement matrix to a point where there was insufficient matrix present in a given unit of volume to properly bind and anchor the fibres. A fifty percent increase in unconfined compressive strength of the samples was achieved with the optimum steel fiber content quoted above. Polypropylene fibre addition also resulted in an increase in strength with increasing fibre content. Strength gains here exceeded those obtained with steel fibres by moderate to high amounts.

Stress-strain measurements indicated that the presence of fibres also increased the residual strength after failure. This would mean that if failure occurs, the backfill would still be capable of supporting considerable load. In practice, this could lead to reduced failures of exposed backfill and less dilution, even when the strength of the material is exceeded.

6.7.4.1 Fibre costs

Although the technical potential for the use of steel and polypropylene fibres in backfill is good, an economic analysis done by Wittreich (1988) showed that their use in optimum quantities is not economically feasible. A similar analysis by Li and Mitchell (1988) included the savings in cement consumption as a result of increased strength due to fibres. They concluded that the savings in cement possible for a given backfill strength by adding fibres would pay for 10,000 commercial mesh elements per cubic meter of fill. This quantity of fibres is sufficient to produce equivalent strengths obtained with the quantity of cement which could be bought for the same price and therefore, reinforcing elements would be only marginally cost effective.

One alternative for reducing costs would be to add the reinforcing elements only in certain layers of backfill and not throughout the entire pour, thereby reducing the overall consumption. Another option would be to study fibres with alternate shapes and compositions. For example, the use of wood chips in some backfills might produce a strengthening effect at a much lower cost (Wittreich 1988).

6.7.4.2 Potential applications

The high cost of presently available reinforcing elements precludes their economical use in large pours of backfill. Therefore, their application should be limited to partial reinforcement along selected areas of the backfill. For example, it has already been suggested that the elements could be placed in selected thin layers of large backfill pours. It is also possible that their use could be limited to the surface layers of mucking floors for cut-and-fill stopes. This would provide extra stability and reduce dilution from scraping buckets and would employ a lower cement content than conventional mucking floors.

6.8 LAYERED BACKFILL, AN INNOVATIVE DESIGN APPROACH

6.8.1 Background

Stratifying backfill pours into alternate layers containing more and less cement is not a new concept. In fact, cemented cut-and-fill stopes often include a top 0.5m thick layer of more richly cemented backfill to function as a working floor and as strong roadways to support heavy mobile equipment. In the mid-1970's, new bulk mining techniques such as blasthole or vertical retreat mining resulted in very large secondary stopes leaving high free-standing faces of backfill from previously filled primary cut-and-fill stopes. This resulted in high free faces of backfill showing a series of successive thin bands of richly cemented fill (i.e. the former cut-and-fill working floors) separated by thicker zones of regular sand/cement backfill mix. The stratification pattern was repeated every 3 to 4 metres. The high fill exposures consisting of layered backfill performed well and produced little sloughing and low dilution. When any sloughing did occur, the extent of the slough zone was limited by the presence of the strong richly cemented bands which acted as barriers.

Upon observation of the good performance of the layered backfill faces, some mines opted to maintain the pattern in subsequent fill pours regardless of stope height. For example, when Falconbridge Ltd.'s Strathcona Mine executed its first pillar recovery by vertical retreat mining in the mid-1970's, the large void was backfilled with cemented sandfill in a ratio of 30:1 sand to cement, with a one foot layer of 8:1 sand to cement poured every nine feet (Lang et al 1977).

6.8.2 Centrifugal Model Studies with Layered Backfill Designs

The results of centrifuge model studies undertaken as a subcontract to this project indicate that cement cost savings of up to 50% might be realized for low strength fills by using a layered backfill design.

Without exceeding 5% by weight (20:1) normal portland cement content in backfill, layered techniques can extend the stable prototype height of a fill face from 20m to 40m. When the width of the exposed fill is less than the fill height and the wall rocks are sufficiently rough to interlock with the fill mass, the stable prototype height of a 20:1 layered backfill model can exceed 80m. Mass failures in some layered centrifuge models could not be produced at centrifuge speeds producing stress similitude with prototype 100m high fills.

The effect of fibre reinforcement of the high strength fill layers was also considered. The fibres gave some improvement in overall strength but not sufficient to justify their expense.

The effect of wall roughness was investigated by testing half the models under smooth wall conditions and the other half under rough wall conditions. The smooth wall models developed no significant side wall friction during failure, and represent the two dimensional condition of an infinitely long fill. The rough wall condition models the case where the stope walls are sufficiently rough to prevent fill boundary movements. In all cases the exposed height to strike length ratio (H:L) is 1.5.

The model scale factor was designed to be about 100. This was based on homogeneous 22:1 tailings to cement backfill (4.55% cement). Thus in the models, 50mm of bulk weak cemented fill is equivalent to a 5m prototype layer thickness and the 10mm and 5mm thick strong cemented fill is equivalent to 1m and 0.5m prototype layer thickness.

The overall cement usage in the layered models varied from about 3.5 % to 5.2% . Thus the failure heights were not expected to differ significantly from the 20m failure height derived from centrifuge testing of the homogeneous 22:1 fill.

6.8.2.1 Model results

The rough walled models were expected to fail at up to 50% higher scale factors in comparison to the smooth wall models. This is according to published data and analysis from homogeneous models; for example see Mitchell et. al. 1982. The layered rough wall models however did not fail until well beyond these expected stress levels.

The wide smooth walled models with 50mm thick layers of bulk fill with low cement content, (40:1, 100:1) failed by shear through the base portion. Wide smooth walled models with higher cement contents in their bulk fill, (30:1) failed by propagation of a failure plane, (and possible tension crack) through to the surface. The wide smooth walled models with the 30mm thick bulk fill layers and 100:1 cement content failed by base sloughing followed by mass failure.

The rough wall models generally failed by sloughing (caving of material from between the strong layers) from the base upwards. The rough wall models with fibre reinforced layers would not exhibit mass failure within the limits of the equipment testing speeds. Unreinforced models usually exhibited mass failure extending back about 50mm into the backfill once the caving mechanism had progressed about half way up the model. These failures did not appear to be influenced at all by the model width.

6.8.2.2 Conclusions

The following specific conclusions are obtained from the data;

1. All layered fill systems are superior to unlayered fills having the same overall cement usage.

2. Fills confined by rough walls are much more stable than similar fills confined by smooth walls.

3. The stable height of fill generally increases with the overall cement usage. Although there is considerable variation in the data, there is no clear indication that there is a preferential bulk layer cement content or strong layer thickness.

4. Thicker strong layers and higher bulk layer cement contents appear to give superior results.

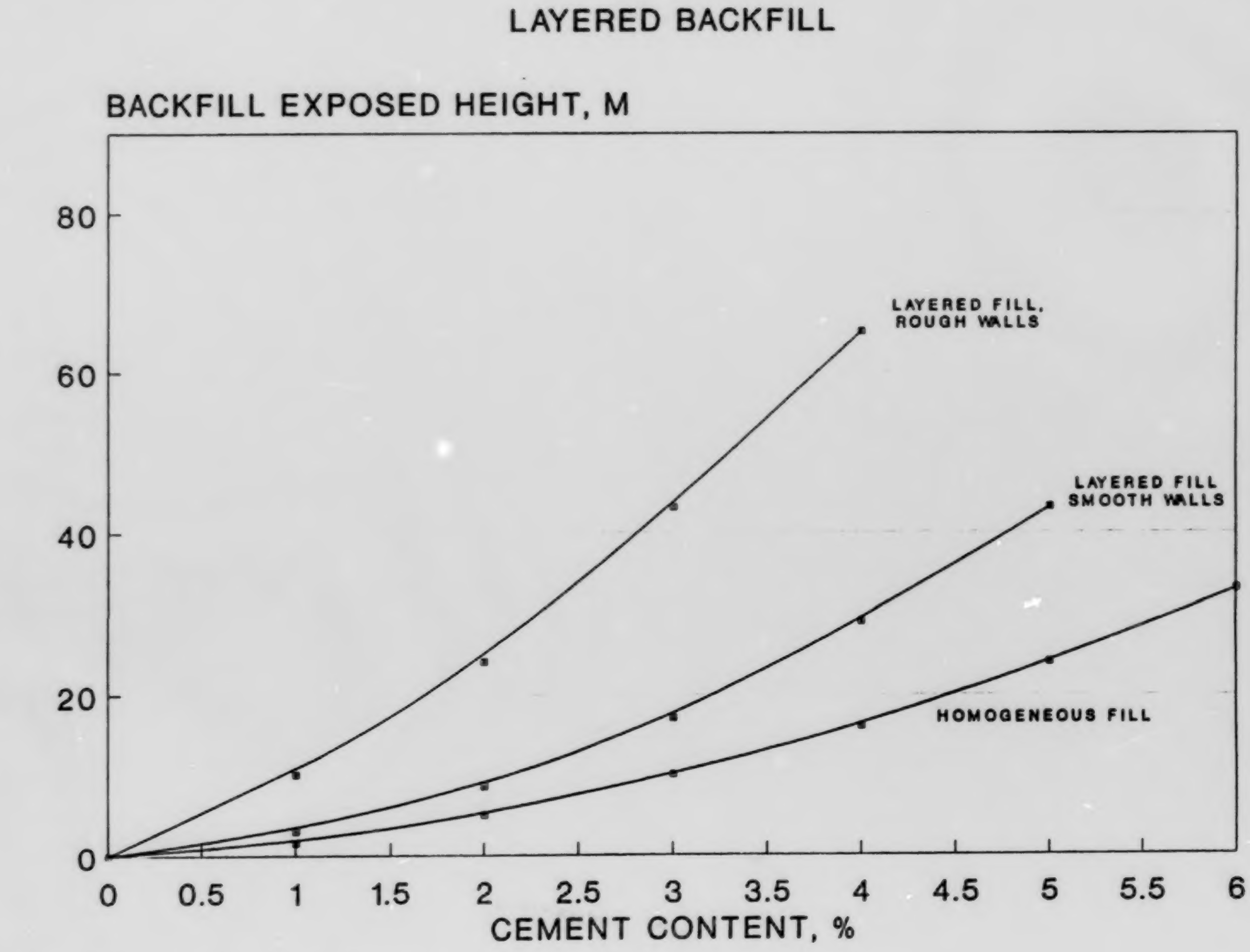
5. If prototype uncemented bulk poured tailings were to drain and densify under self weight stresses so as to have shearing resistance equivalent to laboratory 100:1 mixes, strong layers poured at regular spacings in uncemented bulk pours would be superior to a weakly cemented bulk pour.

Figure 6.19 summarizes the results of the test work. The most surprising result shown is the difference in smooth wall and rough wall critical heights, (an average difference factor of 2.3). This is surprising because previous analytical results which considered shear on rough walls in plain fill (Mitchell, et. al. 1982) indicate only a marginal 25% increase due to rough walls at the height to strike length ratio of 1.5 used in these tests. It has been noted, however, that video and still photographs taken of the tests show that the rough wall condition in layered fill alters the failure mechanism from a single shear plane to a mechanism involving a curved shear plane which exists in the surface of the sample. Indeed, some failures in rough wall models were of an interlayer sloughing type and the fill mass could not be brought down within the limits of the centrifuge. These observations indicate that the combination of wall roughness and strong layers prevents lateral movement, (which may have tended to cause tensile cracking of the strong layers in the smooth wall models) and hence prevents the development of a normal shear plane.

6.8.3 Field Trial of a Layered Backfill Design

In November 1987, Falconbridge's Lockerby Mine in Sudbury was the site of a field trial involving sequential pouring of alternating layers of 3 m of weak (40:1 tailings to cement) and 0.6 m of strong (10:1 tailings to cement) backfill to fill a large blasthole stope. Because of the irregular profile of the empty stope, the tonnage of backfill in the different layers had to be varied in order to maintain the designed thickness of each layer. Each individual layer was allowed to cure for a minimum of two days before continuing with pouring of the next layer. A diagram showing the stope and resulting backfill configuration is given in Figure 6.20. This layer trial resulted in the same overall binder consumption as would be typical of a mass pour design.

Upon concluding the above field trials, it was determined that layered backfill systems offer much merit in large stopes where later high fill exposures would be required. The concept should be studied further in production situations to allow detailed analyses of fill performance and ore dilution through instrumentation and visual inspection.



NOT TO BE USED FOR DESIGN

Figure 6.19, Results of layered fill centrifugal modelling.

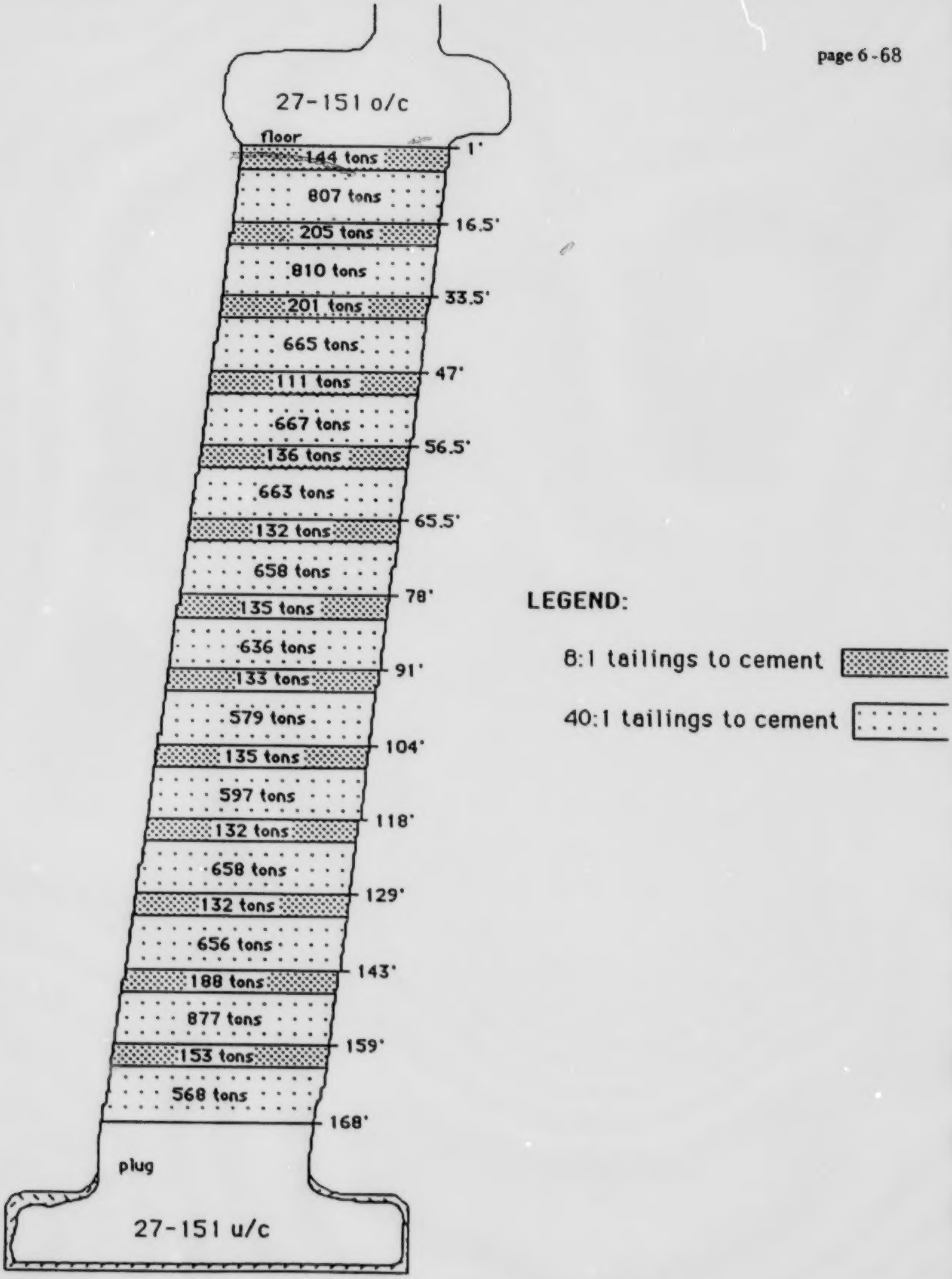

27-151 o/c
floor
144 tons
1'
807 tons
205 tons
16.5'
810 tons
201 tons
33.5'
665 tons
111 tons
47'
667 tons
136 tons
56.5'
663 tons
132 tons
65.5'
658 tons
135 tons
78'
636 tons
133 tons
91'
579 tons
135 tons
104'
597 tons
132 tons
118'
658 tons
132 tons
129'
656 tons
188 tons
143'
877 tons
153 tons
159'
568 tons
168'
plug
27-151 u/c
LEGEND:
8:1 tailings to cement
40:1 tailings to cement

Figure 6.20, Schematic of layered fill trial at Falconbridge's Lockerby Mine.

6.9 SLOPED WALL BACKFILL, AN INNOVATIVE DESIGN APPROACH

6.9.1 Background

The placement of hydraulic fill so that its geometry better matches its ability to stand unsupported has potential for reducing backfilling costs and improving performance. The concept is shown in figure 6.21. The angle of the sloping wall is proposed to be 55 degrees from the horizontal to ensure that ore in the secondary stope would disengage and flow easily along the sloped fill wall into the neck of the funnel for mucking at the drawpoints. The sloped wall of fill should allow less binder consumption for an equivalent height of fill exposure. This concept has been successfully used at the Elura mine in Australia.

6.9.2 Centrifugal Model Studies with Sloped Backfill Wall Designs

A centrifugal model study of sloped wall backfill designs was conducted as a subcontract to this project by Kingston Georesearch Limited.

Theoretical analysis of the slope walled design gave the following critical height, **Hc**, predictions;

1. Plane failure at 45° degrees

$$Hc = 2.84 \ UCS/w$$

where UCS = Unconfined uniaxial strength of the consolidated fill, KPa
w = unit weight of fill, kN/m3

2. Rotational sliding failure

$$Hc = 2.6 \ UCS/w$$

This compares to a predicted critical height for a vertical fill face of;

$$Hc = 2.0 \ UCS/w$$

Chapter 2 discusses the theoretical approach to calculating the stable fill height in more detail. The slope walled fill should be more stable by the factor 2.6 / 2.0 or 1.3.

The actual critical heights achieved in the test models could be described by ;

$$Hc = 7.0 \ UCS/w$$

This is a significant improvement on the predicted results. Figure 6.22 illustrates the difference in the results.

Sloped wall backfill designs show good potential for saving costs and improving stability.

Sloped backfill walls have been proposed at Falconbridge Ltd. for use in large blasthole or VRM stopes to increase the height of free-standing face of fill that can be exposed during mining of secondary (or tertiary) stopes.

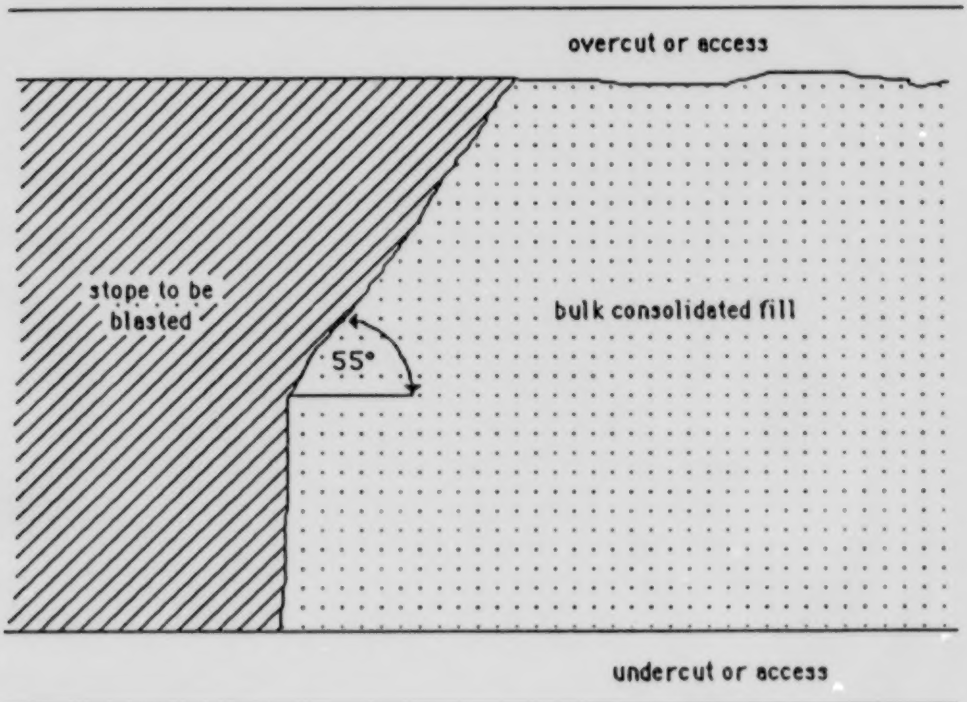

overcut or access
stope to be blasted
55°
bulk consolidated fill
undercut or access

Figure 6.21, Sloped wall fill concept.

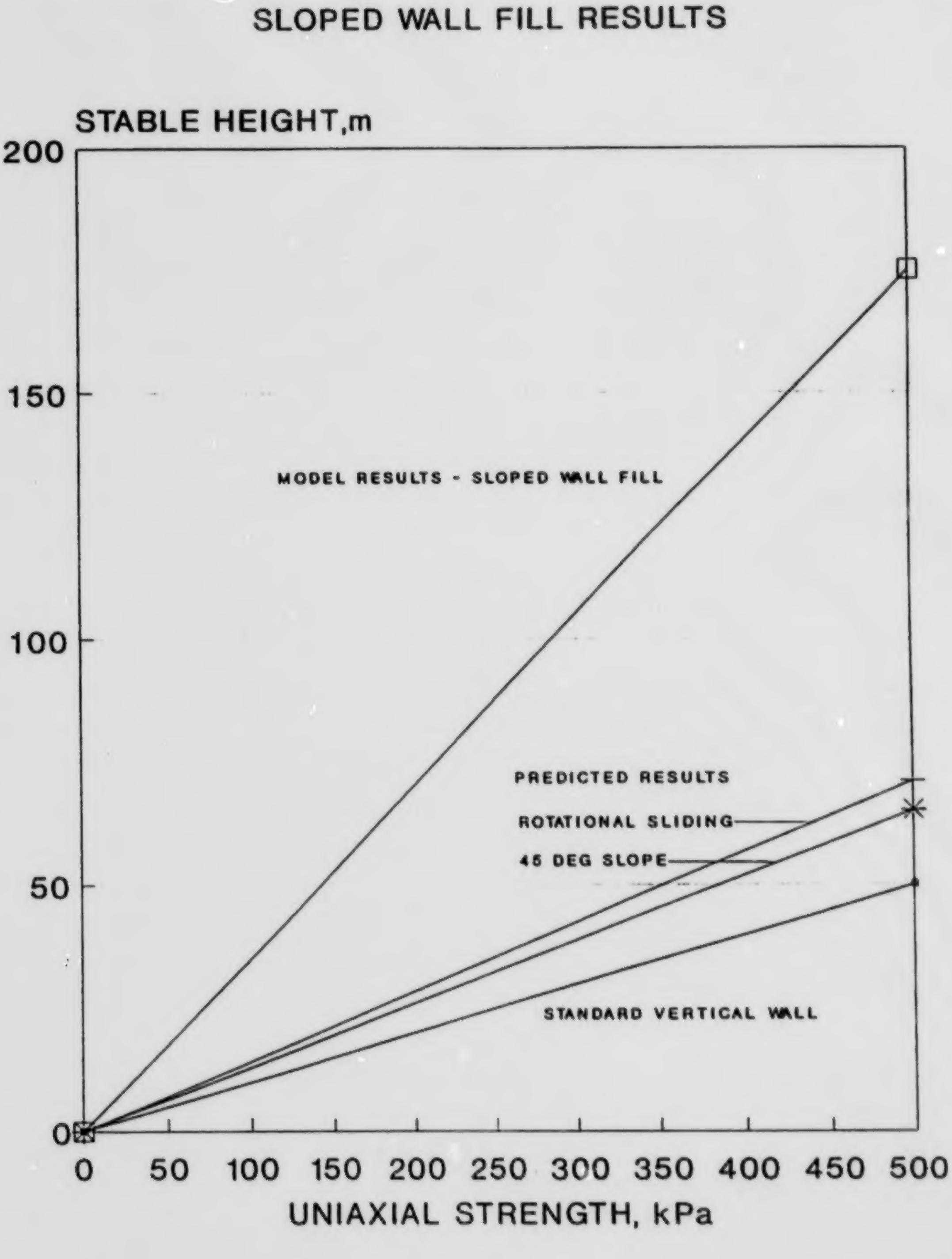


Figure 6.22, Results of sloped wall fill modelling.

6.10 DENSE WALL BACKFILL, AN INNOVATIVE DESIGN APPROACH

6.10.1 Background

A proposed innovative backfill design is the placement of a high strength wedge of hydraulic fill on faces which are to be exposed in subsequent mining operations. The idea is to use the cement in the location where it is needed most; by more efficient utilization, the total cement consumption might be reduced.

A pour site at the upper limit of the stope and at the junction between the primary and secondary stopes would allow a limited volume of high density backfill to attain a shape as shown in figure 6.23. The result would be a retaining wall consisting of a wedge of dense fill which would be thickest at its base. The wall of the wedge facing the secondary stope to be mined would be vertical, while the opposite wall would follow the angle of repose of the dense fill, expected to be 65 to 70 degrees. Once the wall would have set, the remainder of the empty stope would be filled with unconsolidated tailings or sand fill, potentially resulting in significant savings in cement.

6.10.2 Centrifugal Model Studies with Dense Wall Backfill Designs

A centrifugal model study of dense wall designs was conducted by Kingston Georesearch as a subcontract to this project.

The results indicated that the stability of a dense wall design can be adequately described using the free standing wall criteria, as if there was no unconsolidated fill being retained by the dense wall. Hence the critical height, Hc, can be described by;

$$Hc = UCS/w$$

where:
UCS = uniaxial compressive strength, KPa
w = unit weight of the fill, kN/m^3

This is illustrated in figure 6.24. For wall heights greater than ten metres the free standing wall approach appears to be conservative.

It is clear from the modelling that the dense wall design shows great potential for reducing costs and improving backfill stability.

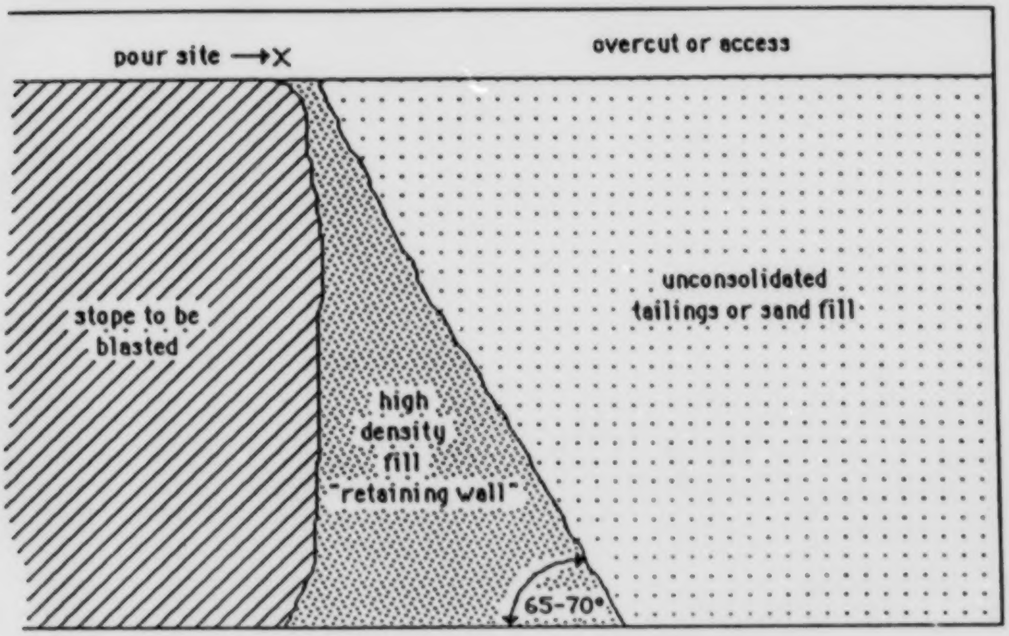

pour site →X
overcut or access
stope to be blasted
high density fill "retaining wall"
unconsolidated tailings or sand fill
65-70°

Figure 6.23, Dense fill wall concept.

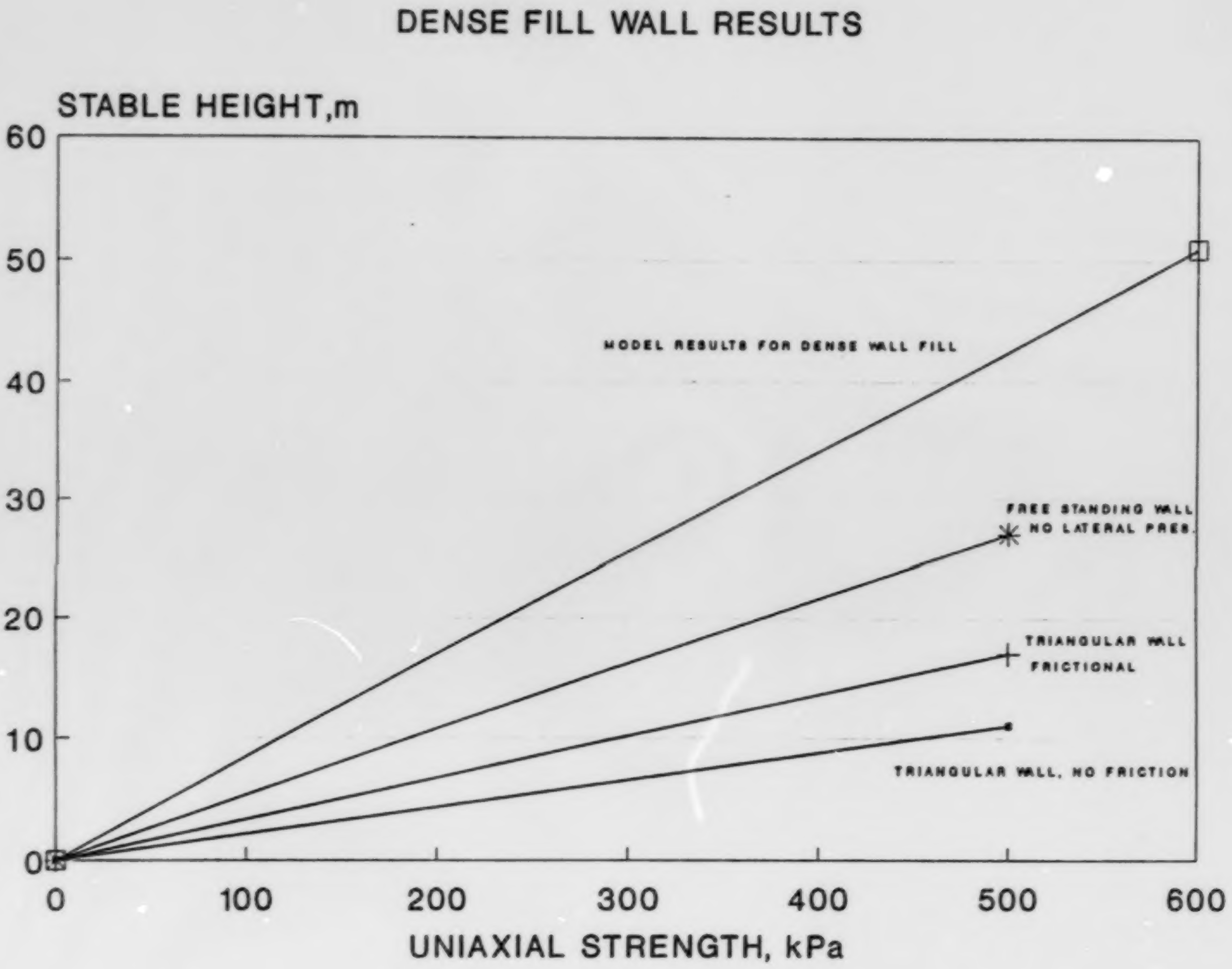


Figure 6.24, Results of centrifugal modelling of dense fill wall design.

6.11 CHEMICAL ADDITIONS

6.11.1 Flocculants

Flocculants are sometimes added to hydraulic fill in an attempt to help decrease the amount of binder and other fine particles which escape with the drainage water. Flocculants used with tailings mixtures are usually "polyelectrolyte" types which consist of high molecular weight polyacrylamides. These heavy molecules carry either a positive (cationic) or a negative (anionic) charge to assist in adsorption onto the electrically charged tailings particles (Fell 1988). This causes the long chain polymer molecule to uncoil and extend itself so that it can attract more tailings particles. Hence, the polymer acts as a bridge between tailings particles and large flocs are formed, which rapidly settle out of suspension.

The primary role of flocculants is to reduce the problems associated with fines in the drainage water, however they can also have other beneficial affects. By helping to keep binder particles in the stope and by also reducing the segregation effects which lead to non uniform binder distribution, it is likely that the strength of the fill will be improved. Drainage characteristics of the fill mass may also be improved by the addition of flocculants.

Even with all the potential advantages mentioned above, flocculants are still not universally added to hydraulic backfill. The advantages are often difficult to measure underground and hence the expense is difficult to justify. Reported work on the use of flocculants and dispersants to improve settled density of hydraulic tailings mixtures indicates varied results which are inconclusive. Mines which have experimented with particle grading to increase settled density by designing an optimal mix of fine and coarse particles have had more success in obtaining consistent and positive results (Thomas et al, 1979). The effect of particle grading on settled density is discussed further in this report under the chapter on hydraulic backfill.

6.11.2 Trials with Chemicals to Aid Tight Filling

For ground control purposes it is often advantageous to minimize ground convergence by filling a stope as close to the backs as possible. The shrinkage of hydraulic fill after placement makes this difficult. It is possible to add chemicals to hydraulic fill which will help minimize this shrinkage. These chemicals are based upon retaining much of the water in the fill, limiting the repacking of the particles which accompanies the exudation of the saturating water. The chemicals also offer other advantages including faster setting time for the binder, less water to be pumped to surface, less slimes released into the mine, and less binder loss which may improve fill strength. A lower density and consequently lower stiffness fill will be placed using these chemicals. The retention of binder in the fill may offset the loss of stiffness due to lower density in practice. Part of the research work of this project aimed to investigate the potential of these chemicals. Work was done at both Falconbridge, Sudbury Operations and Kidd Creek Mines.

6.11.2.1 Trials with Tekfill additive

An additive, called "Tekfill", a custom chemical system of inorganic liquid based on silicate solutions was tried at both Falconbridge, Sudbury Operations and Kidd Creek Mines. Tekfill is added to backfill to accelerate setting time of the binder and to reduce water runoff. Tekfill must be custom designed for each specific application, depending on site-specific backfill chemistry.

The liquid Tekfill is applied into the hydraulic fill line just prior to discharge into the stope. As a result, solid particles are suspended in a thickening slurry until the binders set. The major effects are a reduction in water, binder and slimes escaping from the fill, and a more rapid setting time (Technik Mining, undated). The chemical is sensitive to water quality.

A trial conducted by Technik Mining, suppliers of Tekfill, for Falconbridge, Sudbury Operations on the performance of Tekfill with Strathcona mill classified tailings showed that the water retention was increased but that compressive strength was reduced. The binder tested was ordinary portland cement. Table 6.3 below gives the results.

	Control (no Tekfill)	1.25% Tekfill added
Strength (KPa)		
1 day	298	119
3 days	371	170
7 days	898	430
28 days	1032	473
Water Retention (% of original)		
2 hours	64.5	88.00
4 hours	60.90	88.50
6 hours	60.90	88.70
12 hours	60.90	88.70
24 hours	60.90	88.70

Table 6.3, Results of tests on 8:1 Strathcona Mill tailings fill with Tekfill and portland cement.

The Tekfill was successful at retaining water, thus implying less shrinkage, but at the expense of compressive strength. Test work carried out by Kidd Creek Mines confirmed the effect of Tekfill on compressive strength. A blend of portland cement and flyash was used as the binder for these tests, so the results are not strictly comparable but they are indicative of what could be expected. Table 6.4 below gives the results.

	Strength Development, MPa		
Curing Time	Control (no Tekfill)	1% Tekfill addition (by weight of binder)	2% Tekfill addition (by weight of binder)
28 days	0.70	1.20	0
56 days	0.87	1.56	0.43

Table 6.4, Strength of sandfill test cylinders, 8:1, 68% pulp density.

These results suggest that addition rates lower than 1% Tekfill are beneficial to fill strength. However excess addition rates will cause the fill to have very low strength. This has important implications for the practical implementation of the additive.

A larger scale underground trial was carried out at the Strathcona Mine by Falconbridge, Sudbury Operations. Tekfill was added to regular 8:1 portland cement bound hydraulic fill in a test pour. The Tekfill was introduced into the fill line 100m from the pour point. The amount of drain water was observed for comparison to regular hydraulic fill pours. The following conclusions were made from the test;

1. There was very little drainage water from the pour, indicating that the Tekfill was working well at retaining water.

2. Average shrinkage of the fill was about 20mm, which compares favorably with a normal pour where about 150mm would be expected.

3. After 4 days the pour had not set properly. Strength development was very slow, suggesting that the addition rate was too high and a loss of strength as seen in the lab was occurring.

6.11.2.2 Trials with Sodium Silicate and Hydroset

Kidd Creek Mines were responsible for testing two alternatives to Tekfill. These were Sodium silicate and Hydroset (a product of Grace Chemicals Limited). Results for these test were similar to those of the Tekfill trials, the water retention could be obtained but at the expense of compressive strength if critical addition rate was exceeded. These tests were conducted with a blend of portland cement and flyash (50:50)

Strength Development, MPa

Curing time	Control (no additive)	1% Sodium Silicate	2% Sodium Silicate	1% Hydroset	2% Hydroset
28 days	0.71	1.05	0	0	0
56 days	0.87	1.50	0	0	0

Table 6.5, Strength of sand fill cylinders , 8:1, 68% pulp density.

6.11.2.3 Conclusions

From the test work described above it appears that chemical additives can be successfully used to limit the shrinkage of hydraulic fill so that better tight filling can be achieved. However the addition rates must be carefully controlled or the fill strength may be dramatically reduced or never develop. Control of chemical addition to the degree of precision required in the underground environment would require some careful design.

CHAPTER 7

MONITORING AND INSTRUMENTATION

7.1 Introduction

- 7.1.1 Instrumentation and monitoring trials as part of this project

7.2 Methods for installing instrumentation in backfill

7.3 Instrumentation

- 7.3.1 Stress monitoring
- 7.3.2 Measurement of in situ physical properties
- 7.3.3 Deformation measurements
- 7.3.4 Measurement of sloughing in exposed fill faces
- 7.3.5 Vibration monitoring
- 7.3.6 Visual inspection

7.4 Case studies

- Case 1. Fill dilution measurements during production blasting in a VCR stope
- Case 2. Monitoring stress changes and stope wall deformation in rockfill
- Case 3. Bast vibration monitoring in rockfill
- Case 4. In situ rockfill testing using a pressuremeter
- Case 5. In situ hydraulic fill testing (1989)
- Case 6. Monitoring of pressures in hydraulic fill during and after the filling of blasthole stopes, application of Gloetzl cells
- Case 7. Experience with in situ testing of consolidated rockfill
- Case 8. In situ hydraulic fill instrumentation (1984)

7.1 INTRODUCTION

Monitoring and instrumentation in backfilled stopes is necessary in order to assess the performance of backfill in place and to allow changes to be made either to the backfill system or to mining methods, leading to improved underground stability. There are five main purposes to instrumentation in backfill;

1. To verify whether the fill in place is achieving its design strength and other designed properties so that it can fulfill its purpose;
2. To monitor changes in fill pillars as they take on the load of regional ground support;
3. To monitor damage from blasting and other mining activities in neighboring areas and hence, to predict the extent of fill dilution in the ore;
4. To evaluate the merits of a new fill type;
5. To provide reliable data for input into computer simulation programs.

Monitoring and instrumentation techniques apply at various levels of the backfilling process, beginning with the initial stage of backfill materials selection and extending through to long-term monitoring of fill structures in place.

1. Monitoring for quality control of primary backfilling materials
2. Plant and process control during the batching process
3. Monitoring during placement
4. In-situ testing and long-term monitoring of backfill in place

Chapter 7 is focused on monitoring and instrumentation of fill already in place.

In general, monitoring and instrumentation in fill can be done to produce data relating to five basic parameters which are important in the assessment of fill performance. These are stress monitoring, deformation measurement, the measurement of blast vibrations carried through fill and the fill's damping effects on them, measurement of the extent of fill sloughing, and the measurement of physical properties of the fill after placement. These measurements can be carried out by various methods which involve either direct instrumentation of the fill or visual inspection, all of which will be described later in this chapter.

7.1.1 Instrumentation and Monitoring as Part of This Project

A considerable amount of work was done under the auspices of this project involving the instrumentation and monitoring of backfill at both Kidd Creek Mines and Falconbridge, Sudbury Operations. The results of this work is presented as case studies at the end of this chapter. Some of this work is closely related to research carried out as part of a previous Falconbridge CANMET joint project, In Situ Monitoring and Computer Modeling of a Cemented Sill Mat and Confines During a Tertiary Stage Pillar Recovery, DSS File No. 06SQ.23440-5-9205.

7.2 METHODS FOR INSTALLING INSTRUMENTATION IN BACKFILL

Most monitoring instruments need to be installed in boreholes drilled through the fill. This poses a particular problem because of the difficulty in drilling through fill and because of the nature of fill material which causes holes of any significant length or diameter to cave as the drill is withdrawn from the hole.

Using an auger drill is probably the cheapest method of providing a borehole for fill instrumentation. Unfortunately, auger drills are limited in the depth of borehole that can be achieved due to deviation of the auger bit. For deep boreholes, diamond drilling is recommended for the accuracy it provides. Diamond drilling has the secondary benefits of providing holes with smoother walls which are ideal for instrument coupling, and of producing a core sample of fill. The core can be analyzed to determine its cement content, subjected to various strength tests. The extent of particle segregation can also be determined from a core analysis.

There are several problems associated with coring backfill. Diamond drills use water as a cooling and drilling fluid. This can be detrimental to fill monitoring because water can significantly change the properties of the fill surrounding the borehole. Water introduced into a borehole can leach out cementing agents, thereby reducing strength and producing additional voids in the borehole area. It is recommended that drilling mud be used instead of water to prevent this from happening. Coring in fill can also be difficult in rock fills, especially where large boulders consisting of hard rock are used. The many harsh interfaces caused by the constant transition from hard rock boulders to the softer matrix composed of cement or other binder can cause deviation or jamming of the drill bit. Attempts at coring backfill with conventional diamond drilling methods usually result in marginal recovery.

The difficulties associated with coring in fill have been addressed by Falconbridge Ltd. in a project aimed towards developing fill coring techniques. As a result of the study, Falconbridge has developed a diamond drilling technique capable of extracting relatively undisturbed core samples of weakly consolidated hydraulic mill tailings backfill. The special drill was very successful and was able to obtain core samples from weakly consolidated backfill which were up to 95 percent intact. The technique was developed as part of a joint Falconbridge, CANMET project, DSS File No. 06SQ.23440-5-9205, In situ monitoring and computer modeling of a cemented sill mat and confines during a tertiary stage pillar recovery.

Another technique for coring in backfill has also been described by Plewes et al (1988). The authors suggest that the technique provides intact ,undisturbed core samples in tailings sand to depths of 100 metres. In this technique, a 3 metre long wireline Christensen double barrel coring machine was modified to incorporate a specially designed adjustable core barrel shoe. Optimal core recovery was obtained when the shoe was extended approximately 0.15 cm ahead of the coring bit. A PVC core liner was inserted into the inner core barrel for each coring run. The liner included a steel spring core catcher behind the coring bit to retain sand samples during retrieval and to confine the samples during handling. A low pressure was maintained for the drilling fluid to prevent washing away the cohesionless material ahead of the core barrel. When the sample was retrieved, the PVC liner was cut and sealed at both ends and was frozen in dry ice. The authors suggest that corrections be included for volume expansion due to in-situ stress release, for volume change when the sample passes from the core shoe to the core liner, and for volume change due to freezing.

In regular diamond drilling in backfill, it is possible to resolve the problem of material caving in drill holes after the drill is extracted by installing a steel or plastic liner or casing in the hole to maintain its integrity. This is not the best solution because the physical presence of the casing limits the tests that can be performed in the borehole with any degree of reliability. Another procedure which has been used to prevent hole caving is to drill the hole, grout it up with a cement bentonite grout containing a hydration agent and then re-drill the hole through the grout. This allows the formation of a smoother and more stable borehole wall. The use of drilling mud which was suggested previously in order to prevent the detrimental effects of water in a drill hole can also improve the condition of the borehole wall by preventing caving and by providing a smoother surface for instrument coupling.

Theoretically, it is necessary that the material immediately surrounding the borehole wall is at an in-situ condition to provide representative measurements. Even without the use of casings or grout in the borehole, it is difficult to obtain a true in-situ condition within the hole. The creation of a borehole cavity inevitably results in a redistribution of stresses surrounding the borehole. This causes a squeezing action and the development of a plastic zone in deep boreholes. The roughness of the borehole wall can also influence borehole measurements. It is impossible to obtain a truly representative data when instrumentation is installed in boreholes but for lack of better methods, such data is acceptable to help assess the performance of backfill in-situ.

7.3 INSTRUMENTATION FOR BACKFILL MONITORING

7.3.1 Stress Monitoring

Monitoring of stress fields in backfill can be accomplished through the use of stressmeters and borehole pressure cells. Stressmeters are devices which have the capability of simultaneously measuring stress states and material properties such as in-situ strength, elastic coefficients and fracture, creep characteristics in irregular ground. They also provide site characterization data to assist in the development of FEM computer models (Serata Geomechanics Inc., 1989). Some stressmeters incorporate built-in data processing capability, thereby allowing instantaneous graphic analysis and display using laptop computers at the testing site.

A stressmeter consists of a probe which is inserted in a pre-drilled borehole. The probe contains a loading cylinder which can be pressurized within the borehole. The borehole deformation and fracture patterns which result from the pressurized probe are measured in relation to the pressure loading. Resolution can be as low as one micron. A typical stressmeter probe measures slightly over two metres in length and weighs approximately 22 kg. More details are available in Appendix I.

Borehole pressure cells operate on the same principle as the previously described stressmeter. It consists in fact of a cylindrical, inflatable, elastomeric membrane which can be sufficiently pressurized inside a borehole to cause failure of the borehole wall. In brittle formations, failure appears in the form of a pair of mutually perpendicular radial fracture planes, whereas in ductile media, it is seen through plastic yielding of the borehole boundary. The instrument permits long-term monitoring of stress states and in-situ creep. It is described further in Appendix I.

A name often mentioned in backfill monitoring is the Glötzl pressure cell. This is a hydraulically pumped, uni-directional stress measuring device that requires a hydraulic load cell, hydraulic feed and return lines as well as a pump. The feed line is pressurized until the pressure delivered at the pump equals that in the load cell. The highest steady reading achieved when the fluid is allowed to return to the pump is then measured. The pressure delivered in the pump equals the pressure acting on the cell. Details and a sketch of this instrument are provided in Appendix I. Glötzl cells are suitable for measuring stresses in backfill as well as in intact rock.

7.3.2 Measurement of In-Situ Physical Properties

The measurement of physical properties of backfill material in place can be achieved using such devices as the stressmeter borehole pressure cells and an instrument known as the propertymeter.

A propertymeter is an instrument which is similar to a stressmeter except that instead of a loading cylinder, it employs eight penetrometer pistons deployed in two mutually perpendicular orientations in symmetry. In theory, this enables eight penetration tests to be done simultaneously on the borehole wall, resulting in a truer characterization of material properties in complex ground such as backfill. A computerized system automatically collects, analyzes and graphically displays the data on a laptop computer while measurements are taking place (Serata Geomechanics Inc. 1989). Although the propertymeter's main function is to measure physical properties, it can also be used to determine the stress state of the medium. More details on this instrument and the method of data analysis are given in Appendix I.

A pressuremeter, made up of a pressure cell as described in section 7.3.1 above, is recommended by Falconbridge personnel as an effective instrument to determine in-situ engineering properties of backfill. The procedure involves inserting a pressure cell into a bored hole and expanding it hydraulically. The stiffness of the backfill, its modulus of elasticity and Poisson's Ratio can be calculated from the pressuremeter test results.

One instrument which has previously been used in soil mechanics testing has recently been adapted for use in in-situ backfill testing by a group in Québec (Scoble et al 1987). The Pencel Pressuremeter is a simple and economic tool that can be used in combination with electronic piezocones or static cone penetrometers to give data such as pore water pressure, strength, modulus of deformation, limit pressure, shear modulus, etc. The instrument can be inserted in a pre-drilled 35 mm diameter hole or, when backfills are below 65 MPa in stiffness, it can be hydraulically pushed into the backfill mass using a hydraulic ram and a string of insertion rods, eliminating the need for drilling. In addition to eliminating drilling expenses, inserting the pressuremeter by pushing through intact backfill also provides testing under less disturbed conditions. Tests using this instrument can be conducted up to depths of 10 metres and at any desired interval during insertion. For example, the probe could be inserted to a 10 metre depth with tests taken every metre or every half-metre. Each test takes approximately 20 minutes. Details and reports of various field trials using the Pencel instrument are reported in Scoble et al (1987).

7.3.3 Deformation Measurements

The amount of deformation to which a backfill mass is subjected when under loading can be measured using instruments such as various types of extensometers, strain cells, borehole deformation gauges and closure rate meters.

The extensometer is by far the most popular instrument used in Canadian hard rock mines to determine deformation and creep in boreholes. Extensometers can also detect movement in a footwall due to the weight exerted on it by a mass of backfill. A basic wire extensometer consists of a probe housed in an aluminum cylinder which is inserted into the mouth of a borehole. An anchor is placed deeper in the borehole and connected to the probe with a stainless steel wire. A displacement transducer located inside the probe measures the relative deformation between the two components. This instrument is useful for measuring sill deterioration, pillar expansion and stability of a vertical fill face.

There are other types of extensometers such as the Irad gauge sonic extensometer, which is a device used for measuring axial deformation in boreholes. It is inserted in a BX borehole (60 mm diameter) to a maximum depth of 7 metres. The probe emits ultrasonic pulses that are reflected back up the probe. The differences in arrival times of the reflected signals are used to determine distances between reference points. Other extensometers include rod extensometers, magnetic extensometers, vernier surface extensometers and tape extensometers. Each of the extensometers mentioned above is described in more detail in Appendix I.

Other deformation and stress measuring devices offer data by employing an overcoring technique. Overcoring is a procedure by which relaxation of an annulus of rock around a borehole is created thereby permitting measurable strains to be induced in the material being tested. This in turn allows the determination of absolute stress in rock or of stress changes over a long term by the use of stress cells. For example, a 90 mm diameter borehole (NXCU core) is drilled through the medium to be tested. The bottom of the hole is flattened using a flattening bit. A 37 mm diameter borehole (EX core) is then centered in the flattened bottom of the existing hole and extended to the required depth for installation of the measuring instrument. The instrument (usually a stress cell) is then installed into the EX hole to provide a reference strain measurement in-situ. The cell is then cored-out ("overcored") and final strain readings are taken after extraction. The stresses are then calculated from strain changes in the overcored medium.

Borehole deformation gauges are instruments that can be used to determine the state of stress that exists in the backfill surrounding the borehole by using overcoring techniques. These gauges also give measurements of creep in boreholes. One such gauge is the CSIRO triaxial hollow inclusion stress cell. This is a three-dimensional borehole deformation gauge which provides complete stress state analysis from one single borehole. The cells, which consist of electrical resistance strain gauges embedded in an epoxy body, are non-reusable. The cell measures strains induced in the material around a borehole by the relaxation of an annulus of rock during overcoring. When establishing a depth within a borehole at which a CSIRO stress cell can be installed, the core extracted from the overcoring hole can be examined for integrity. Glue is used to provide a bond between the gauge and the borehole wall in the best location as indicated by the core. This instrument is most suitable for use in competent materials. Details are given in Appendix I.

The "doorstopper" strain cell is an instrument that operates on the same principle as the CSIRO cell described above with the main difference being that the doorstopper provides two-dimensional stress measurements only. Here, the strain gauges are mounted on the end of a plastic holder which is glued to the flattened end of a borehole. The zero strain reading is taken and then the cell is overcored and retrieved. Two dimensional in-situ stress is calculated from the change in strain readings. A complete three dimensional stress determination can be achieved with this device by instrumenting three non-parallel holes. Details on this instrument are provided also in Appendix I.

The CSIR triaxial strain cell, like the CSIRO hollow inclusion cell, is a three-dimensional borehole deformation gauge. When this strain cell is inserted into a hole, strain gauge pistons are pneumatically forced against borehole walls. Initial readings are taken and the cell is overcored for removal. Final strain readings are then taken and the stress state can be determined. Details are available in Appendix I.

In all three of the above instruments, it is essential that experienced personnel be present to install the gauges and cells properly and to ensure that the instruments are suitably positioned and coupled to the borehole wall. The success of all three of the methods is greatly dependent on the quality of the bond produced between the gauges and the borehole wall by the glue being used. Hot and humid conditions often preclude their use.

One instrument which does not require a glue bond between the gauge and the borehole wall is the USBM borehole deformation gauge. This gauge consists of a spring-loaded caliper which physically measures change in the internal diameter of a borehole to allow stresses to be determined. The caliper incorporates strain gauges so that this instrument can be installed in a borehole and overcored to determine stresses in a manner similar to the previous three instruments. To obtain a complete determination of the stress state, three non-parallel boreholes must be instrumented. This instrument is also detailed in Appendix I.

Closuremeters are instruments which can be used to measure the rate of convergence of one exposed face with respect to an opposite face. These usually consist of spring-loaded potentiometric devices which are anchored to two reference points where relative movement is to be measured. Examples of such instruments are the Vernier closuremeter, the stope closuremeter and COM closuremeter. All are described in Appendix I.

7.3.4 Measurement of Sloughing in Exposed Fill Faces

It is important to be able to measure the amount of sloughing which occurs in fill faces as they are exposed to blasting activities in stopes that are being mined immediately adjacent to them. Such measurements give an indication of the performance of open fill faces and also help in determining the amount of fill dilution of the ore being mined. Tests commonly conducted to determine the extent of sloughing of fill faces include profiling using the USBM Sonic Probe, measurements using sloughmeters and the "flashlight on a rope" test.

The USBM Sonic Probe profiler is an instrument which has been tested at Falconbridge Ltd. to determine the extent of sloughing occurring along open faces of backfill. This is a sonic focussing device with a focal length of approximately 10 m. It is used to profile the dimensions of voids created by blasting against a backfill face by lowering the probe through production blastholes in active VRM stopes located adjacent to previously backfilled stopes. The void profile which is measured after each production blast can be correlated with known stope dimensions and other measurements of dilution to indicate the extent of backfill failure along the face. The data can also be correlated with sloughmeter readings and "flashlight" tests, as described below.

Sloughmeters are simple devices which can be manufactured in-house and which indicate failure in specific locations in exposed backfill faces. A sloughmeter consists of a wire secured in backfill through which an electrical current is passed. Failure in backfill material by sloughing causes the wire to break, thereby creating an interruption in current flow through that particular wire.

A flashlight test can also be done to determine the extent of sloughing in a backfill sill. This is a very crude test which consists of lowering a flashlight on a rope through an inclined borehole in the fill. The light from the flashlight is visible from the collar of the hole as long as it is inside the borehole. When the light disappears, the flashlight has reached the end of the hole. Any shortening of the hole length from one time to the next indicates sloughing of the fill along the bottom of the hole.

7.3.5 Vibration Monitoring

Vibration monitoring in backfill relates mostly to the measurement of the propagation characteristics of blast vibrations as they travel through backfill. The effect of blast vibrations on backfill is one of the most important factors in fill design. Changes in all other measurements taken during instrumentation trials such as closure rates, extent of sloughing, etc. should be related not only to stress states in the surrounding formation and within the fill itself, but also to the fill's response to dynamic loading caused by blast vibration.

The most popular instrument used for measuring the rate of attenuation of blast vibrations as they travel through the rock / backfill interface and through the fill itself is the geophone. Geophones allow determination of peak particle velocities and frequency characteristics of vibration energy propagating through the fill as a result of nearby blasting. At Falconbridge Ltd., the preferred geophone is a triaxial accelerometers capable of measuring P and S waves in three directions. Triaxial accelerometers can be inserted in boreholes in backfill. From the accelerometer readings, an attenuation curve for the dissipation of energy waves in fill can be developed.

The method of placement of accelerometers in fill is very important in determining their performance in terms of amplitude and frequency analyses. Coupling of the accelerometer with the borehole wall is even more important for proper frequency analysis. Proper coupling can only be achieved by providing continuity in acoustic impedance between the medium and transducer. Otherwise, the sensor has to be in physical contact with the backfill. Epoxy resin glue is sometimes used to secure accelerometers in position. Some prefer grouting in a perforated plastic pipe casing in the hole. This latter technique is not recommended if the accelerometers are being used to provide information to conduct spectral analyses.

7.3.6 Visual Inspection

Boreholes that have been drilled in fill can be inspected visually using borehole cameras, fibre optic devices or an instrument called a petroscope. These instruments allow the observer to inspect boreholes for any signs of fracturing or deformation. Records are kept of each examination so that subsequent ones can be compared to earlier examinations to allow the effect of time on the condition of the backfill immediately surrounding the borehole to be assessed. Although the borehole camera is much more expensive than the other two instruments in this category, it automatically produces permanent records of all test runs whereas the other two rely on the operator's expertise in note keeping for future reference.

Very small diameter video cameras are available for borehole inspections. Boreholes must be 60 mm or more in diameter to enable the use of such cameras. The videos taken provide a permanent record of the state of the material immediately surrounding the borehole at any given time and can be used at various time intervals to determine long term changes. A schematic and description of one such video camera called the Rees Borehole Camera are given in Appendix I.

A fibre optic instrument called a stratascope can also be used for direct inspection of boreholes in backfill. This instrument consists of a fibre optic bundle that transmits light to the end of the stratascope. The image of the illuminated area is transmitted back along the instrument to the eyepiece for viewing. Although the stratascope does not automatically produce a permanent record, an experienced operator can hand-produce a reasonable record for future reference. The stratascope is described in Appendix I.

The petroscope is an instrument consisting of an inclined mirror and light source which can be positioned at various depths within the borehole. A telescope at the borehole collar allows the viewer to inspect material conditions around the borehole. The viewer must produce his own record by hand as this instrument does not automatically produce any record. This instrument is also described in Appendix I.

7.4 CASE STUDIES

CASE 1: Fill Dilution Measurements During Production Blasting in a VRM Stope:

As part of this project Falconbridge Ltd., Sudbury Operations conducted a trial of the USBM Sonic Probe. This was used to determine the amount of sloughing on adjacent fill walls during mining of a pillar by vertical retreat mining at the Lockerby Mine in Onaping. This was done by profiling the mined-out void after VRM decks using the USBM Sonic Profiler as described previously in section 7.3.4. Stope profiles were taken at overcut floor elevations corresponding to minus 70 feet, minus 50 feet and minus 30 feet. Void profiles were measured after blasting decks 3, 4 and 5. Deck 6 was the sill deck and therefore, it was not profiled. Hence, three profiles were taken at minus 70 feet, two at minus 50 and one at minus 30. One ITH drillhole in the center of the stope was selected as the measurement hole.

Results showed inconsistency in readings repeated at the minus 70 foot elevation. The cause was attributed to the sonic head having different coordinates each time a reading was repeated due to the inclination of the borehole and the reduced length of the hole with each successive blast. Therefore, the profile at minus 70 feet is not considered reliable. Profiles taken at the 50 and 30 foot elevations are considered to be more accurate. Figures 7.1 and 7.2 show the results of the measurements.

In this study, profile measurements provided an added bonus in that the blast designer used the information to design each successive blast. For example, after deck 3 was fired, the sonic profile indicated that there had been damage to a rock wall adjacent to the stope being mined as a result of the proximity of drillhole No. 9. This caused some dilution of the ore with waste rock. As a result, the blast designer decided not to load hole 9 for deck 4 in order to prevent further dilution from occurring. Also, holes 3 and 4 were dropped in deck 4 because of evidence in the sonic probe measurements that damage to the fill wall had occurred as a result of these holes firing in the previous deck blast. The profile also showed an improvement in stability of the rock wall as a result of hole 9 having been dropped. In conclusion, the sonic probe measurements not only gave an indication of fill sloughing for the purpose of of assessment of fill performance, but they were actually used to optimize the quantity of explosives consumed and to reduce dilution of the ore.

The above was a simple experiment which provided quick feedback on blasting effects on fill walls adjacent to an active stope. More detailed experiments could be carried out to study the effect of blasting parameters such as charge weight per delay, firing sequence, drill hole layout and other parameters on the extent of fill wall damage. This could be accomplished by conducting various tests simultaneously on one active stope which would involve instrumentation such as sloughmeters, accelerometers, USBM Sonic probe, pressure meters and others described herein.

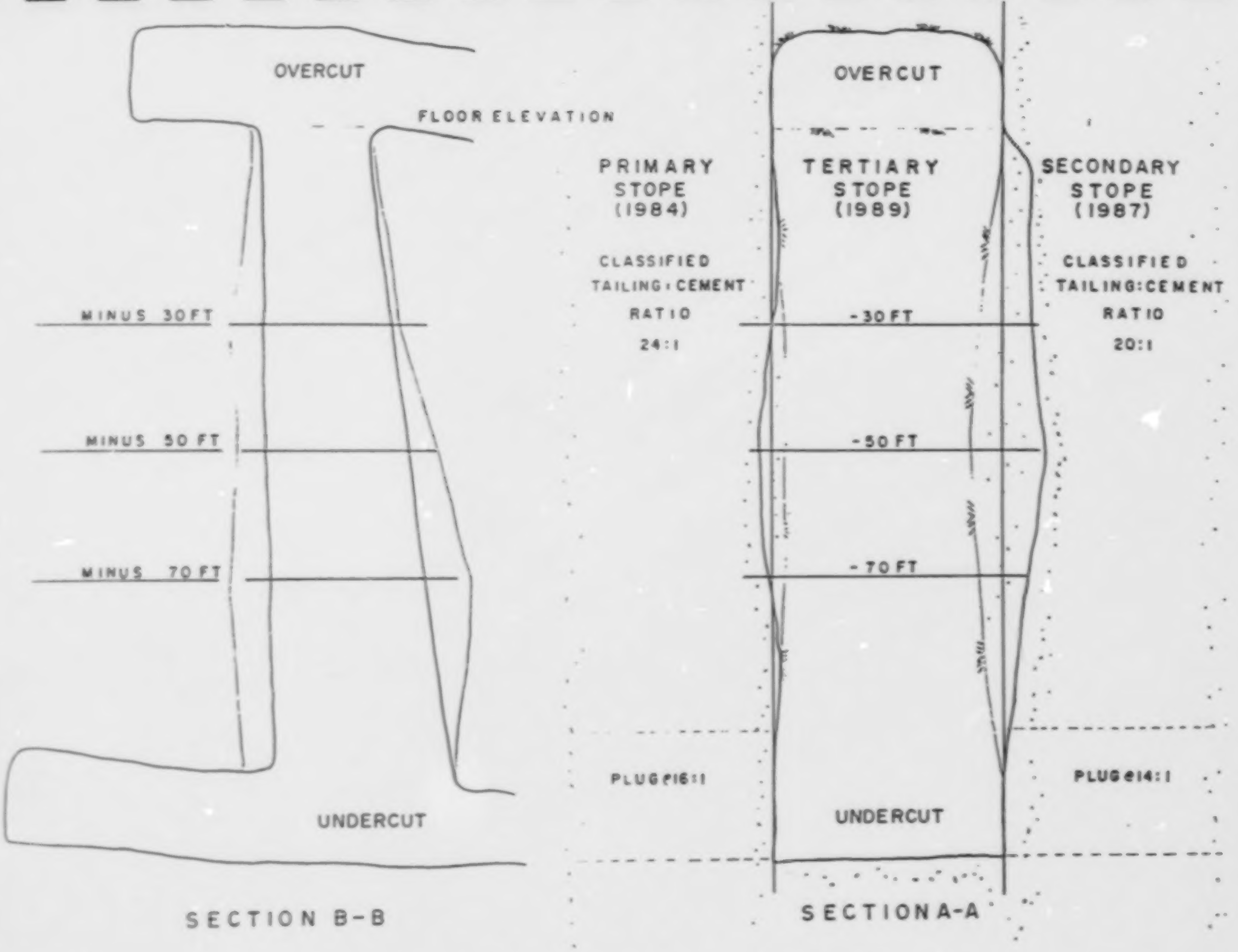

OVERCUT
FLOOR ELEVATION
MINUS 30FT
MINUS 50 FT
MINUS 70 FT
UNDERCUT
SECTION B-B
PRIMARY
STOPE
(1984)
CLASSIFIED
TAILING : CEMENT
RATIO
24:1
PLUG@16:1
OVERCUT
TERTIARY
STOPE
(1989)
-30FT
-50FT
-70FT
UNDERCUT
SECTION A-A
SECONDARY
STOPE
(1987)
CLASSIFIED
TAILING:CEMENT
RATIO
20:1
PLUG@14:1

Figure 7.1 Profile of stope measured with USBM sonic profiler.

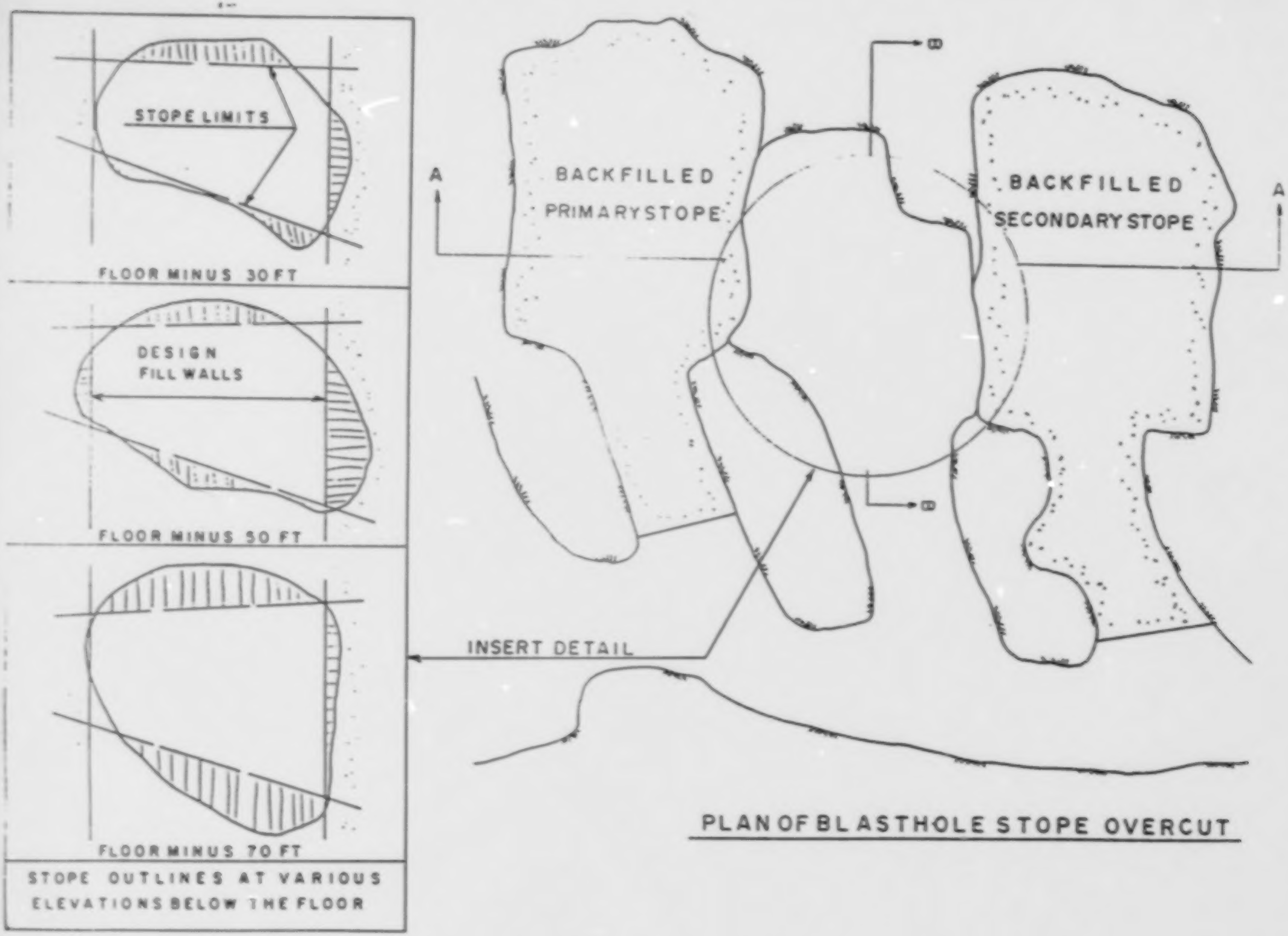

STOPE LIMITS
FLOOR MINUS 30 FT
DESIGN
FILL WALLS
FLOOR MINUS 50 FT
FLOOR MINUS 70 FT
STOPE OUTLINES AT VARIOUS
ELEVATIONS BELOW THE FLOOR
A
BACKFILLED
PRIMARY STOPE
B
B
BACKFILLED
SECONDARY STOPE
A
INSERT DETAIL
PLAN OF BLASTHOLE STOPE OVERCUT

Figure 7.2 Plan of stope measured with USBM sonic profiler.

CASE 2: Monitoring Stress Changes and Stope Wall Deformation in Rock Fill: Part of this project was to carry out a backfill monitoring program . The primary 40-641-ST stope at Kidd Creek No. 2 mine was selected as the location for this monitoring program using extensometers and Irad gauges. Monitoring of this stope started after it was completely mined out. The adjacent stope, 40-631-ST, was also monitored while being mined and filled. The surrounding rock was instrumented to determine rock stress changes, rock movement and seismic characteristics of the rock and consolidated rockfill.

Figures 7.3a,b and 7.4 shows the stope layouts and instrument locations. Two Irad gauges (I-24 and I-25) were used to measure the horizontal rock stress changes. The Irad stressmeter installed into a 3.7 cm diameter diamond drill hole was used to measure stress change in the surrounding rock as a function of borehole deformation. Rock movement was measured using two borehole extensometers, one at each side of 40-641 (GE-13) and 40-631 (GE-14) stopes. A third extensometer to measure rock and fill movement could not be installed due to low RQD of the rockfill. For exact orientation and anchor points of the Extensometers refer to Appendix B.

Wall Stress Measurements (40-641 & 40-631 stopes): The results of the stressmeter monitoring program are presented in tables 1 to 3, Appendix B. In these tables, the values were converted to stress change using the stressmeter calibration equation also provided in Appendix B. Total stress changes versus time in 40-641 and 40-631 stopes are presented in Figures 7.5 & 7.6 respectively. Figure 7.5 shows that while filling 40-641-ST, between days 35 to 130, the ground around the filled stope relaxed about 0.17 MPa. The major inflection in the stress change with time occurred on days 132 and 165, corresponding to the major ring and/or slot blasts in 631-38-1 & 631-38-2 respectively. Appendix B shows the exact date for each blast. After each blast due to the increased stope dimensions, rock stress increased and the maximum compressive stress change at I-24 stressmeter was 0.31 MPa, point F.

A sudden decrease in stress was noticed after the final blast for 631-38-1 X.C., point F-H, Figure 7.5. This was probably showing a shift of the abutment stress field surrounding the stope, from a zone of rock subjected to confining stress to one of stress relaxation. The maximum stress release due to the increased relaxation zone is shown by point H, with a magnitude of 2.43 MPa. Ring and slot blasting for 631-38-2 X.C. from point H to J was associated with an increase in compressive stress on the stope wall. The stope wall started to relax after the final blast at 631-38-2 X.C. Deformation occurring between blasts may have been due to drawing of the ore and also progressive fracturing at highly stressed areas which resulted in redistribution of the wall load. From point J to K, there was no blasting and the mucking of 40-631-ST was ongoing. Filling of 40-631-ST started at day 278 and was completed at day 310. No stress change was observed during this period.

Stressmeter I-25, adjacent to the blasted stope, showed the same trend in stress change measurements as did stressmeter I-24, Figure 7.6. There were noticeable changes in stressmeter readings around days 132 and 165, as shown in Figure 7.5. The increased magnitude of the compressive stresses due to the blasting at days 132 and 165 and the relaxation process after blasting shocks were the only major difference between I-24 and I-25 readings shown in Figure 7.7. The maximum compressive stress change measured by I-25 stressmeter was 3.94 MPa. This stress change occurred around days 130-140, when 10,000 kg of explosive was used for final slot blast at 38-1 sublevel.

Another major slot blast at 38-2 sublevel using 13,000 kg of explosive caused a 2.9 MPa compressive stress change adjacent to 40-631-ST wall, measured by I-25 Irad gauge. After this blast, complete relaxation of the surrounding area took place with the maximum relaxation value being 7.1 MPa at day 168. On the same day, I-24 readings showed the highest relaxation value of 2.4 MPa, which was 33% of the I-25 value. The maximum relaxation values were obtained 2 days after the slot blast.

Filling of 40-631 ST started at day 278 and was completed at day 310. During this period, the wall surrounding this stope relaxed by a magnitude of 1.73 MPa. This could correspond to the amount of stresses that were redistributed to the fill material.

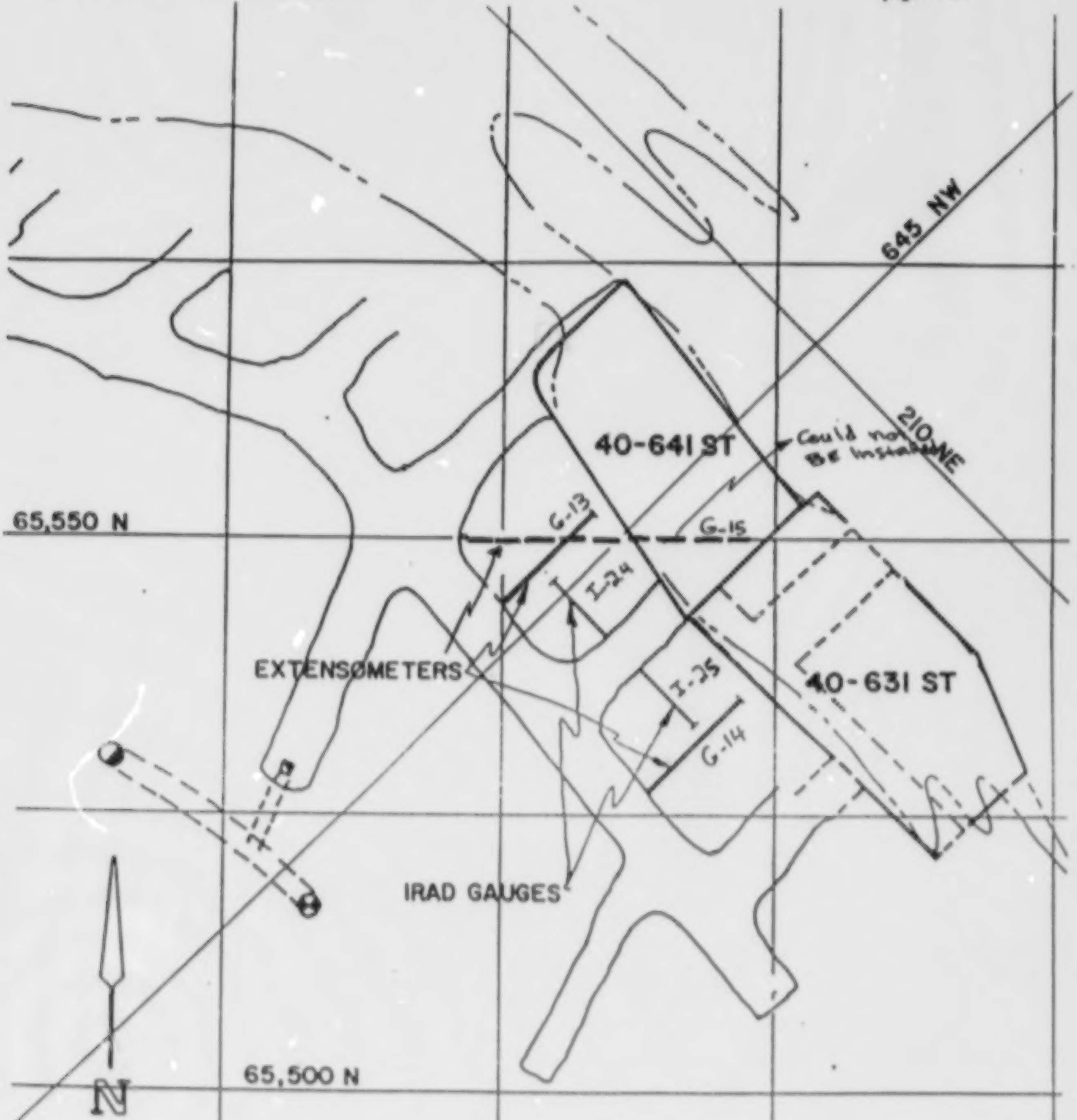

645 NW
210 NE
40-641 ST
Could not Be installed
G-13
G-15
I-24
EXTENSOMETERS
I-25
G-14
40-631 ST
IRAD GAUGES
65,550 N
65,500 N
N
38-1 SUBLEVEL PLAN
65,700 E
65,750 E
1:500

Figure 7.3a Location of extensometers, Kidd Creek 40-641 stope - plan section.

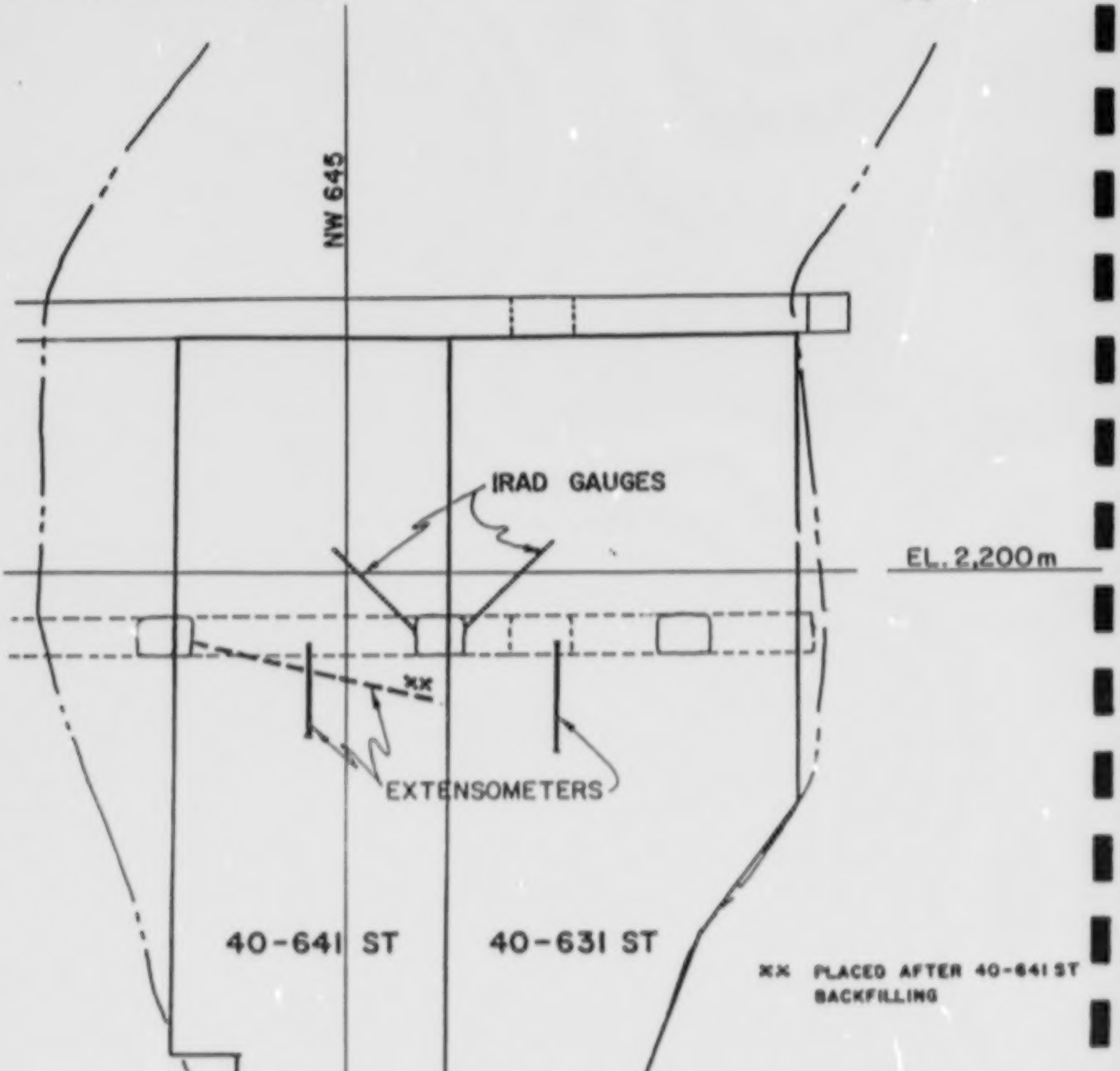

NW 645
IRAD GAUGES
EL. 2,200 m
XX
EXTENSOMETERS
40-641 ST
40-631 ST
XX PLACED AFTER 40-641 ST BACKFILLING
EL. 2,150 m
SECTION LOOKING N-E

Figure 7.3b Location of extensometers, Kidd Creek 40-641 stope - longitudinal section.

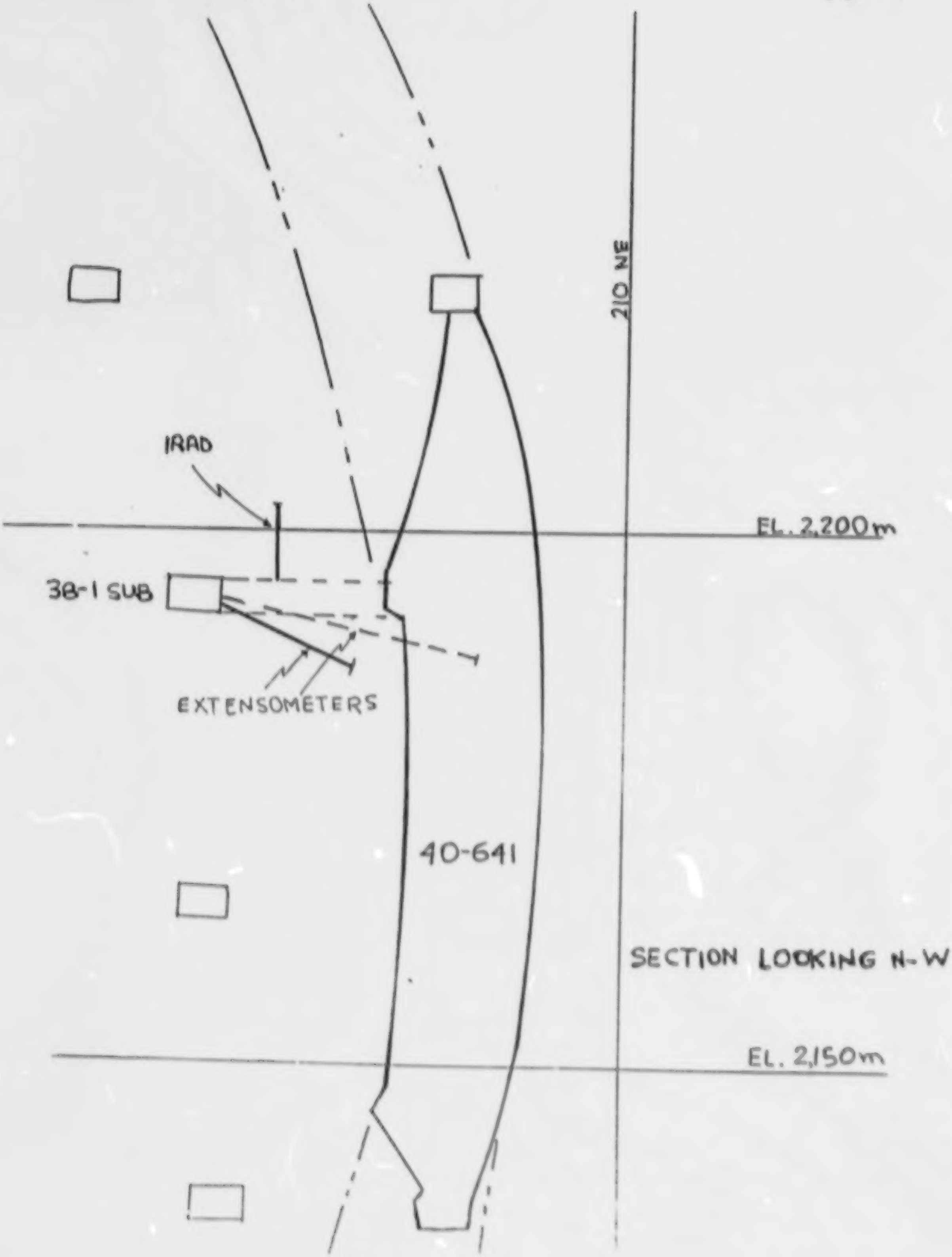
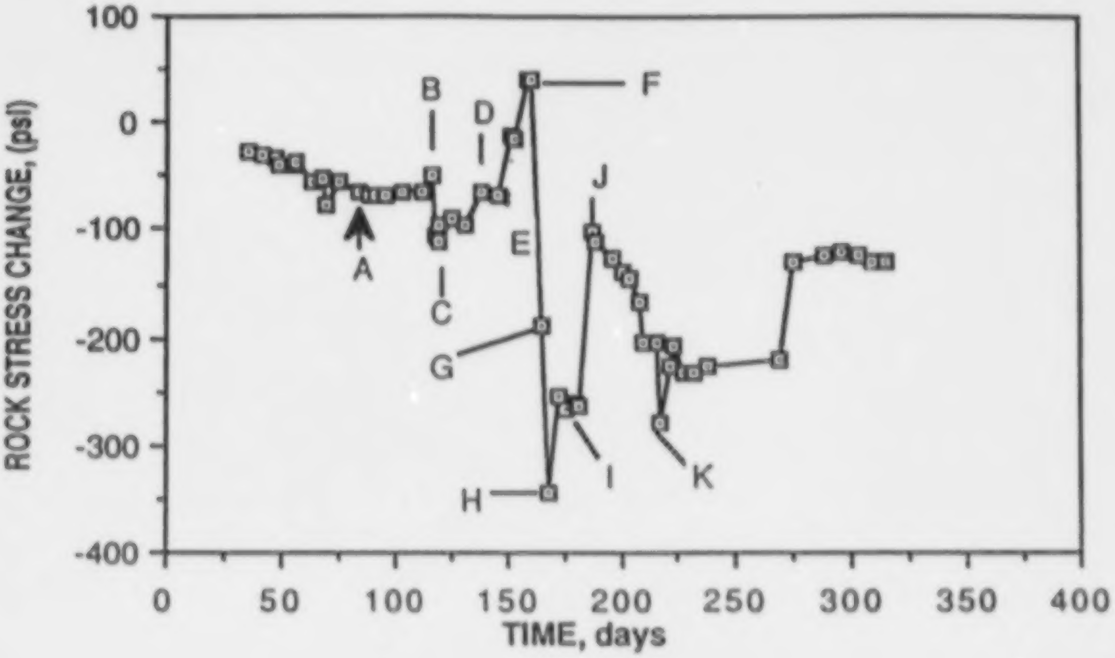

210 NE
IRAD
EL. 2,200m
38-1 SUB
EXTENSOMETERS
40-641
SECTION LOOKING N-W
EL. 2,150m

Figure 7.4 Location of extensometers, Kidd Creek 40-641 stope - typical transverse section.



A: blasting started

Amount of explosive used, Kg

A-B: 5825

B-C: NO BLASTING

C-D: 1625

D-E: 10,125

E-F: 6,950

F-G: 5,625

G-H: 2,675

H-I: 13,725

I-J: 2,800

J-K: NO BLASTING

NOTE: In stressmeter readings, positive stress change indicates compression and a negative stress change indicates relaxation, tension.

Figure 7.5 Stressmeter readings from I-24 installed in 40-641 stope.

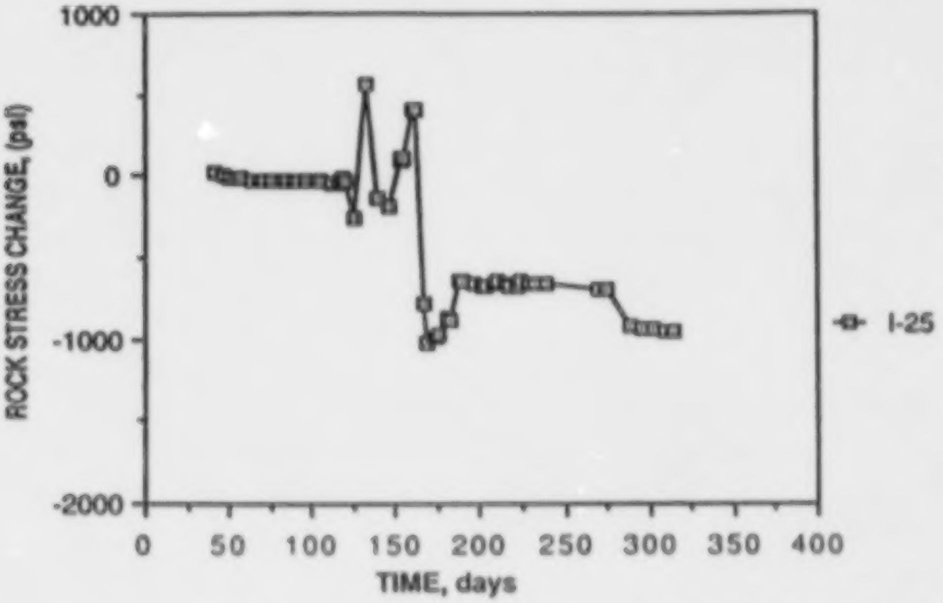


Figure 7.6 Stressmeter readings from I-25 installed in 40-631 stope.

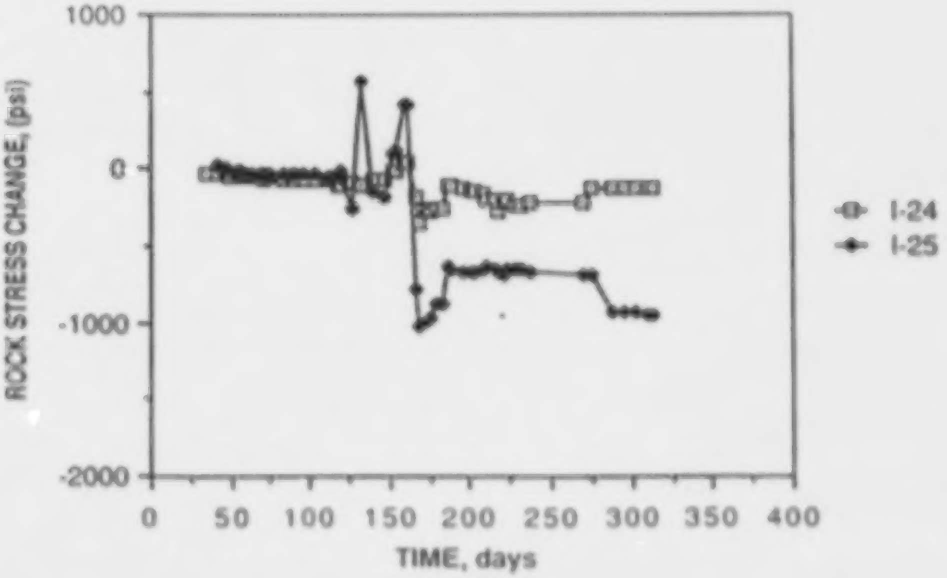


Figure 7.7 Stessmeter readings I-24, I-25 combined.

Pillar Stress Measurements (28-671 stope): In another case study at KCM, the change in pillar stress adjacent to the stope to be filled was monitored. This study showed the effect of fill on the stress distribution at the adjacent pillar, since no production blasting was conducted in the vicinity of the pillar during the monitoring period. The pillar had been virtually isolated from the lateral ground stress, as its two vertical walls had been separated by the placed backfill and the open stope. Two Irad stressmeters were installed in the pillar to measure the stress change in vertical and horizontal components as backfill was being introduced into the open stope.

As shown in Figure 7.8, both vertical and horizontal components indicate a slow response in the early stages of filling which continued to increase in the post fill stage. The vertical component appeared to respond faster than the horizontal one, indicating the effect of confinement as a result of the decrease in the height to width ratio of the pillar as depicted in Figure 7.9 (Yu, 1987).

Wall Deformation Measurements (40-641 & 40-631 stopes): The total displacement versus time for G-13 and G-14 extensometer readings are presented in Figures 7.10 & 7.11 respectively. As expected, the stope wall expanded adjacent to the mined-out area and contracted in the solid ground. Figure 7.10 shows that the stope wall relaxed with a slow rate up to day 104, point B, at which point blasting started. After each blast, expansion continued throughout the wall opposite to the blasted area. Most of the recorded expansion in 40-641-ST took place at the time of major blasts in 40-631-ST . The maximum expansion in 40-641-ST without blasting would have been under 1mm. The blasting of 40-631-ST increased this value to 3.2 mm (ch.#3), measured by G-13 extensometer. The deformation occurring between blasts was due to drawing of the ore in 40-631-ST and also progressive fracturing of the the ground in the highly stressed zones. The sharp slope of the line between points C and F, Figure 7.10, corresponded to the major slot and ring blasts at 631-38-1 and 631-38-2 sublevels. After starting to fill 40-631-ST at point G in Figure 7.10, the line had the same slope as the one before blasting started. The wall expansion before blasting started was around 2.5×10^{-3} mm/day and it increased to 4.1×10^{-2} mm/day between points C & F. The expansion rate during blasting was around 17 times the rate prior to blasting.

The maximum expansion measured by G-14 extensometer was approximately 33 mm, between points H and I. Figure 7.11 indicates that about 80% of the total deformation happened between points B-G, which was during blasting and mucking of the stope. The slope of the line in this period is around 0.34 mm/day, as given in Table 5, Appendix B. The G-14 results indicated that wall deformation related to mining of the stope included a period of;

1. Short period of compression
2. Rapid expansion caused by blasting
3. Slow time-dependent expansion related to ore drawing and mining.
4. Additional expansion after the fill has been placed and the mined-out area is extended.

Extensometer G-15 which was planned to monitor the movement through backfill during mining and filling of 40-631-ST, could not be installed due difficulties encountered in drilling through consolidated rockfill.

Wall Deformation Measurements (838 stope): In previous studies, measurements of wall deformation in the footwall of 838- ST was conducted throughout its backfilling process over a four-month period. Two wire extensometers were used to measure the deformation in this stope, as shown in Figures 7.12 & 7.13. Extensometer M-1 was located at the edge of the stope toward the adjoining rib pillar, and M-2 was installed opposite to the middle of the stope, normal to the wall face. Measurements had been taken throughout the filling period, and Figures 7.14 and 7.15 show the measured data. No production blasting was taking place in the vicinity of the stope and therefore, the measured data showed the sole effect of consolidated rockfill on wall deformation.

The measured record revealed that, on commencement of backfilling in the stope, the outward displacement of the footwall halted and inward movement began for a contraction of 0.9 mm in the wire of M-2. This continued over the entire backfilling period. The longest wire in M-1 also detected a contraction of 0.4 mm. These results clearly indicated that the placed consolidated rockfill developed an active pressure state compressing the relaxed stope wall. (Yu, 1987).

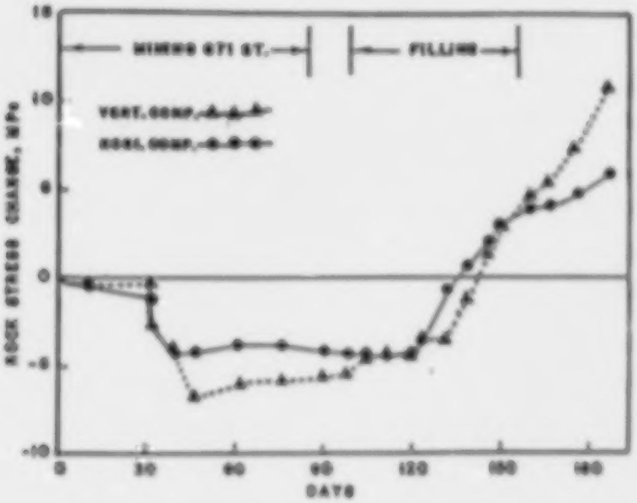


Figure 7.8 Measured rock stress change vs stope mining and filling with consolidated rockfill. (Kidd Creek Mines 28-671 stope)

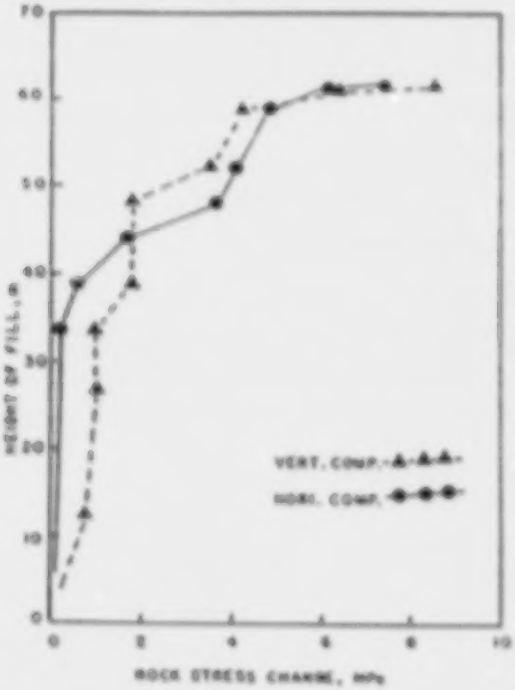


Figure 7.9 Fill height and rock stress change. (Kidd Creek Mines 28-671 stope)

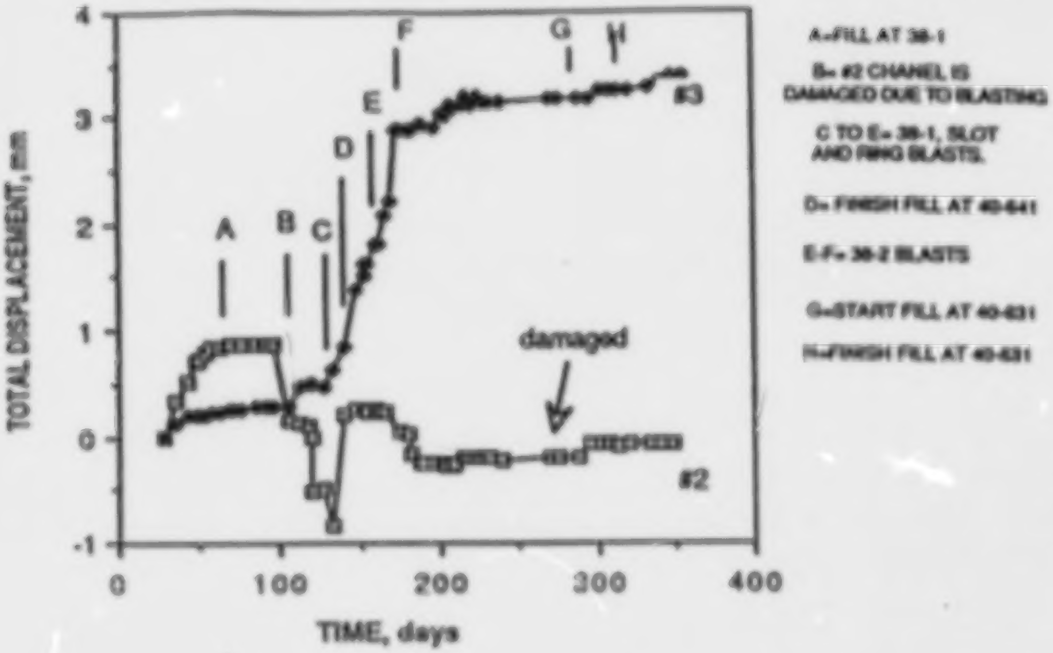


Figure 7.10 Movement measured in extensometer G-13. (40-641 stope)

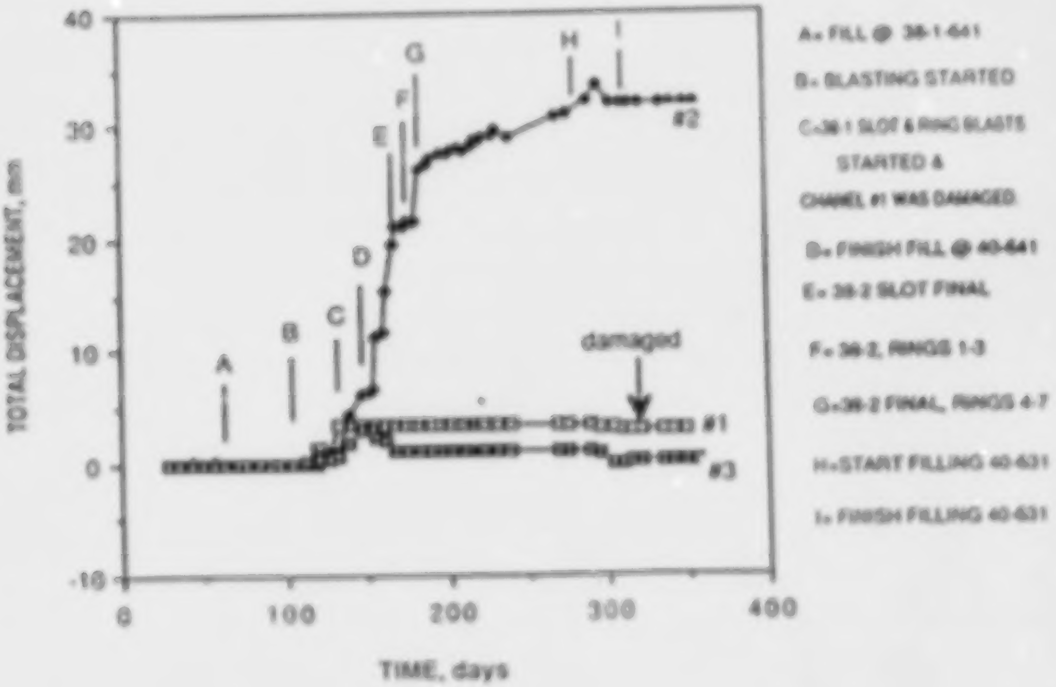


Figure 7.11 Movement measured in extensometer G-14. (40-631 stope)

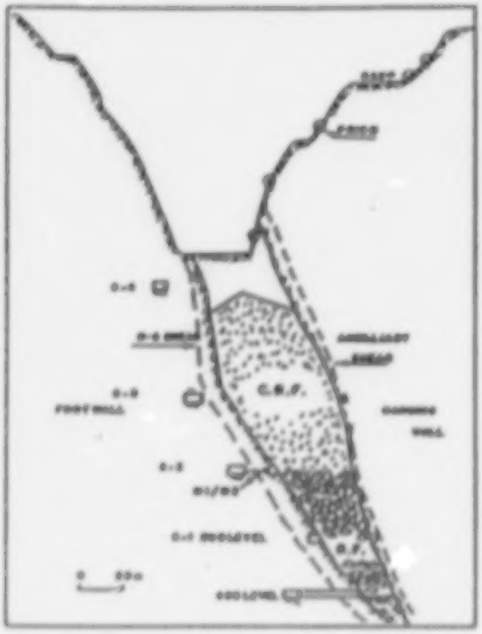

FOOTWALL
HANGING WALL
C.R.F.
R.F.

Figure 7.12 East West section through 838 stope.

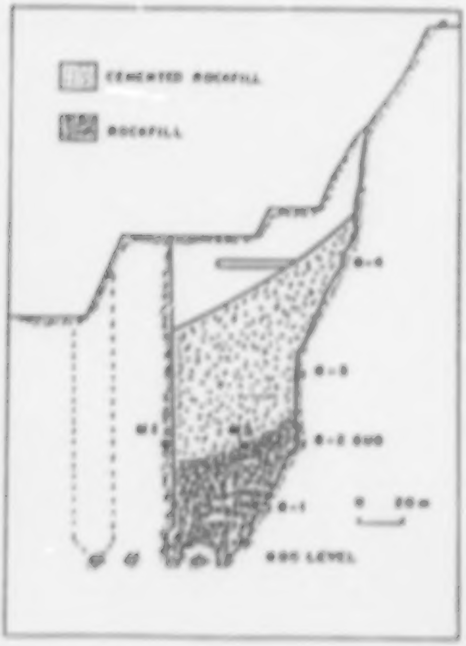

CEMENTED ROCKFILL
ROCKFILL

Figure 7.13 North South section through 838 stope.

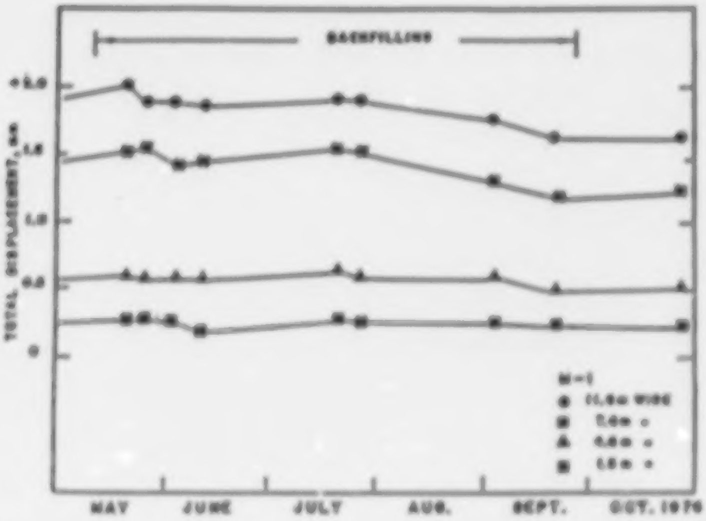


Figure 7.14 Displacement of footwall measured on wire extensometer M-1.

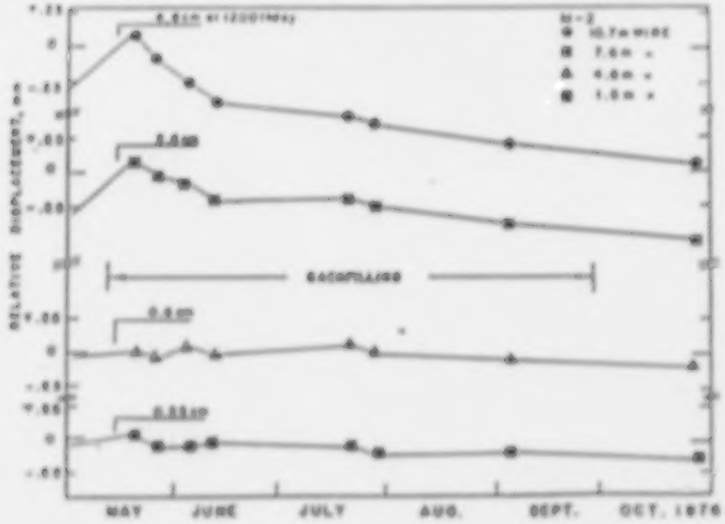


Figure 7.15 Displacement of footwall measured on wire extensometer M-2.

CASE 3: Blast Vibration Monitoring in Rock Fill: When a pillar adjacent to a filled stope is blasted, the fill must be able to sustain not only the gravity loading of the overlying fill material, but also the dynamic effects applied during blasting. Stress waves generated by blasting can be degraded due to;

1. Absorption of seismic intensity from the source, following the inverse square law.

2. Absorption of seismic energy in the medium during propagation according to an exponential decay.

3. Partitioning of the waves at interfaces due to impedance mismatch. This study was carried out to quantify some parameters of seismic wave propagation within a consolidated rockfill mass.

The major dynamic properties investigated in this monitoring program were threefold:

1. **Transmission of seismic waves in CRF**
   To measure the transmission of seismic energy from rock to consolidated rockfill

2. **Attenuation in CRF**
   To measure the attenuation factor in terms of seismic magnitude

3. **Amplitude reduction through backfill :**
   To measure the reduction in seismic energy as the wave passed through rock, fill, and back to rock.

Procedure: The general blast and monitoring locations are shown in Figures 7.16, 7.17 and 7.18. The amounts of explosive employed in blasting were as follows:

- Blast A-----------------1650 kg of Amex (21 holes)
- Blast B-----------------250 kg of Amex (2 holes)
- Blast C-----------------250 kg of Amex (1 hole)

Monitoring was conducted using two sets of triaxial velocity sensors and a seven channel F.M. instrumentation recorder. Analysis of the data was carried out with a signal analyzer.

All the studies in the blast vibration monitoring used the ' peak particle velocity ' (P.P.V.) of a vibration for assessing the performance of the media through which the shock-waves were transmitted. The P.P.V. is the rate of change of displacement and is proportional to the product of displacement and frequency.

Results: Readings in all tables were converted from voltage to velocity values by using the calibration number given for the monitoring instrument. The P.P.V. values were obtained by finding the square root of the square sum of the component readings from the three orthogonal components (Table 6, Appendix C).

BLAST A : This was a stope slot blast, with the amount of explosive used in the blast ranging from 12 kg to 84 kg per delay for a total 1650 kg of explosive. Ground vibration was measured using two sets of triaxial seismometers at a distance from the blast of 51 m in rock and 53.2 m in CRF respectively. The first set of readings (ch. 4-6) was taken after the shock fronts had traveled only through rock. The other set (ch.1-3) showed a reduction in P.P.V due to the waves traveling into fill (Table 6, Appendix C). Results in Figure 7.19 showed that the transmission coefficient for particle velocity from the rock through backfill averaged 37%. The amount of energy transmitted through fill could have been much higher if the fill had had a lower void ratio and a finer size of material at the interface of two media. Past studies at KCM revealed that the transmission coefficient varied in the range of 25% to 73%.

BLAST B: This was a small blast compared to blast A, with a total of 250 kg of explosive used. Both sets of seismometers were installed at the same distance of 50 m from the blast source. The sensor in the fill would detect the shock waves traveling the first 15 m in rock and the other 35 m in CRF. The sensor in rock would pick up the shock waves traveling through rock only. Measured results indicated that the P.P.V. at the fill sensor and the rock sensor were 1.7 i.p.s and 2.8 i.p.s respectively. From these two sets of data, the attenuation of seismic waves in the CRF was found to be 0.42 db/m as shown in Appendix C. This is comparable to the past measured data of 0.57 db/m in competent fill and 0.95 db/m in a segregated zone (Yu, 1989).

BLAST C : Blast C was conducted to study the magnitude of amplitude reduction of shock waves through backfill. As in blast B, 250 kg of explosive was employed for this trial. The energy from the blast traveled through rock, fill, and then back to rock respectively. Blasts A and B were monitored on 38-1 sublevel, but blast C was monitored on 38-2 sub-level. By comparing the magnitude of the wave amplitude in rock before and after traveling through fill, the reduction factor could be obtained. Channels 4-6 were placed in rock and channels 1-3 were also in rock but the signals they read had passed through fill. The measured data, however, are not presented due to inconclusive results.

Discussion: The results obtained from blast A could be used in future designs and results from blasts

B and C, especially blast C, need more detailed investigation. This study showed that a considerable reduction in seismic intensity up to 37% was observed at the interface between rock and CRF and the attenuation of seismic waves in fill was approximately 0.42 db/m. Also, from past studies, it was observed that a competent consolidated rockfill mass was subjected to a scabbing failure at an estimated particle velocity of 30 cm/sec. This vibration level may be considered as a maximum allowable particle velocity in planning a safe production blast against fill. Stope fill failures were seen to be less severe for blast holes drilled in parallel to the fill than those drilled toward it.

In the process of analyzing seismic data, the difficulty in reading the lower limits of seismic traces resulted in trying another approach. The peak amplitude (not peak-peak) of the seismic tracing for each shot, found by assuming sinusoidal vibratory waves, was compiled and presented in Tables 7 and 8 of Appendix C. The data was then used to calculate the transmission coefficient of seismic waves for blast A and blast B. The results matched closely with the peak-peak amplitude readings as shown in Table 6, Appendix C.

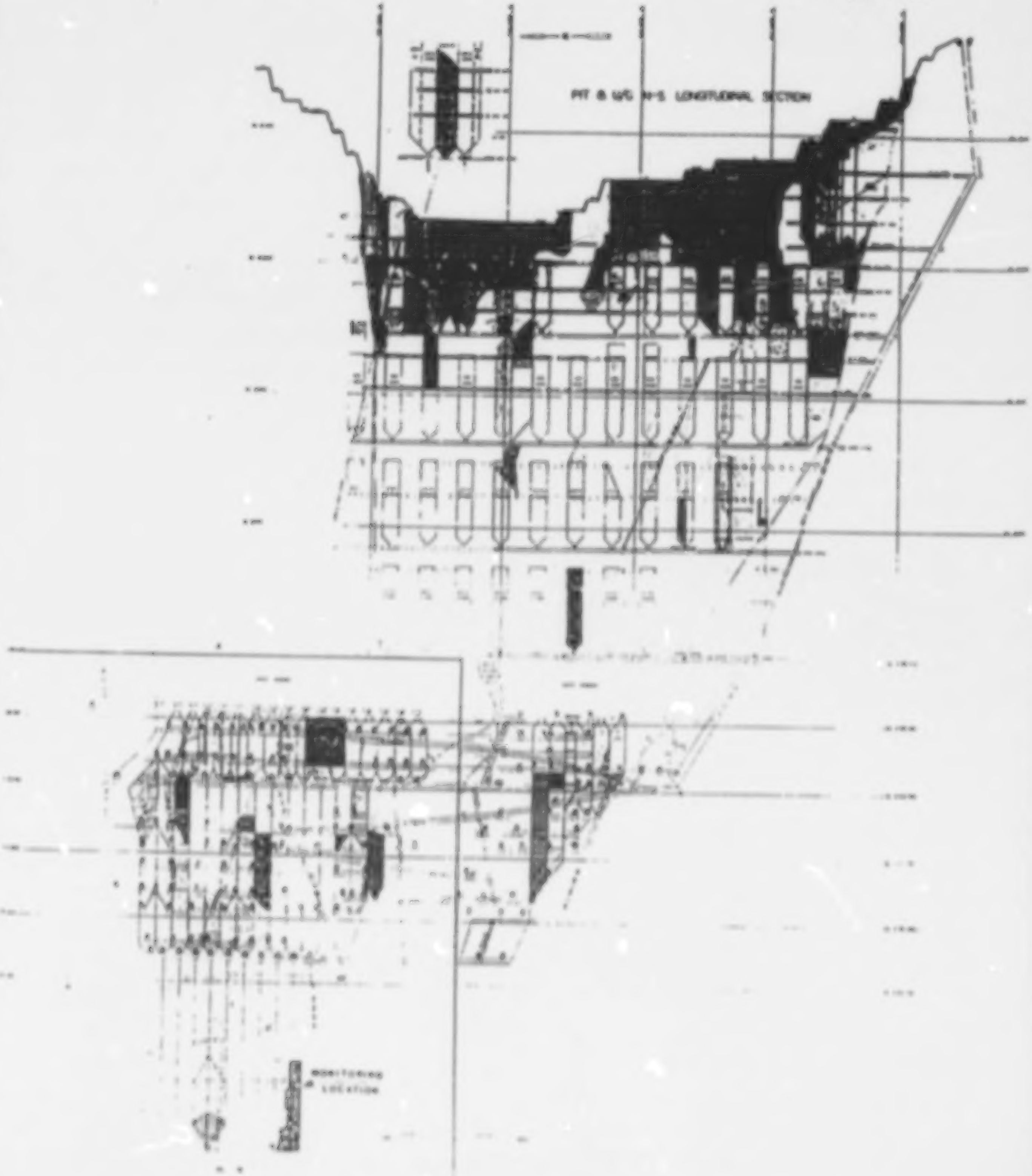

PIT & U/G N-S LONGITUDINAL SECTION
MONITORING LOCATION

Figure 7.16 Section of Kidd Creek Mine showing blast monitoring location.

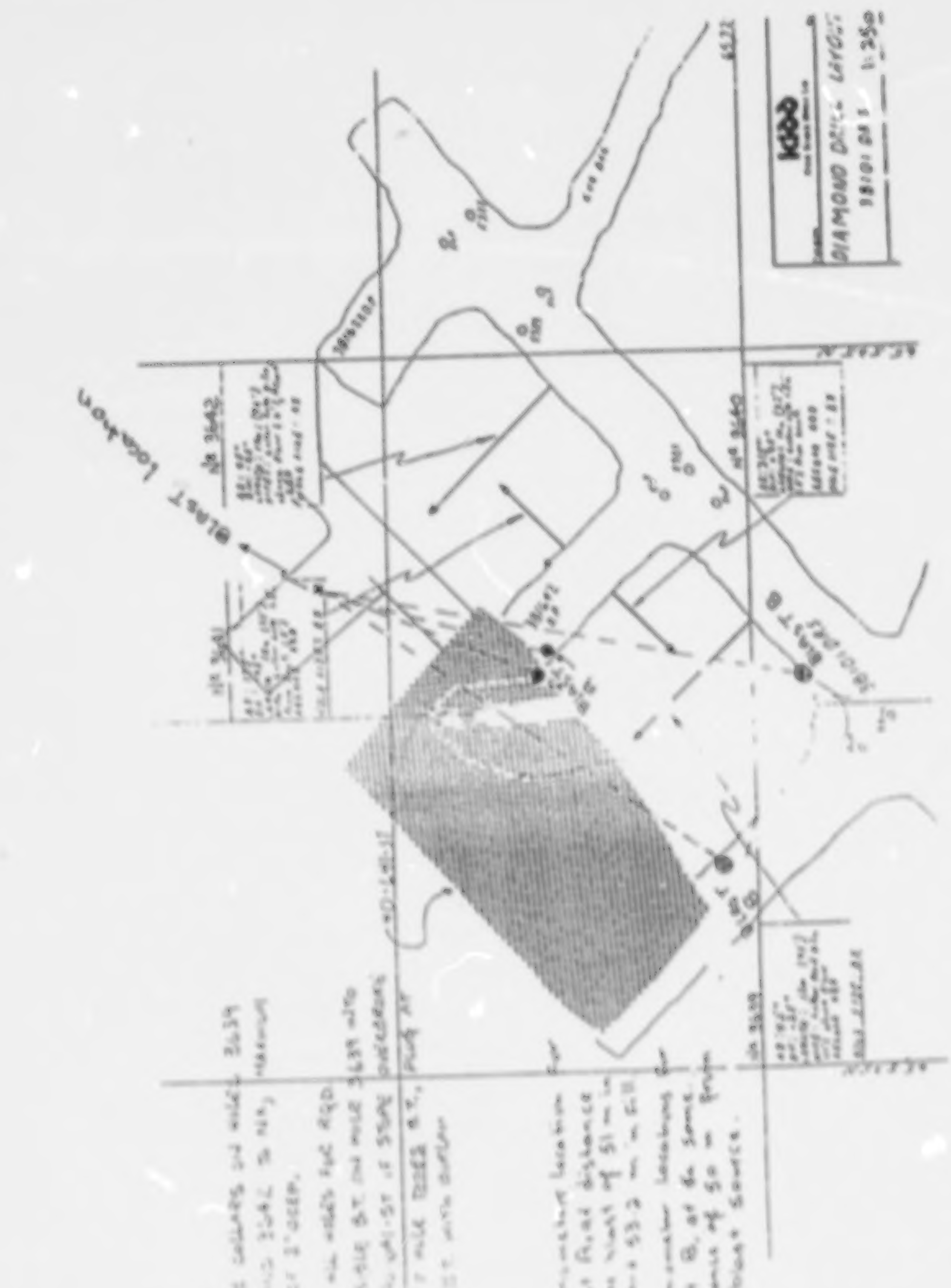
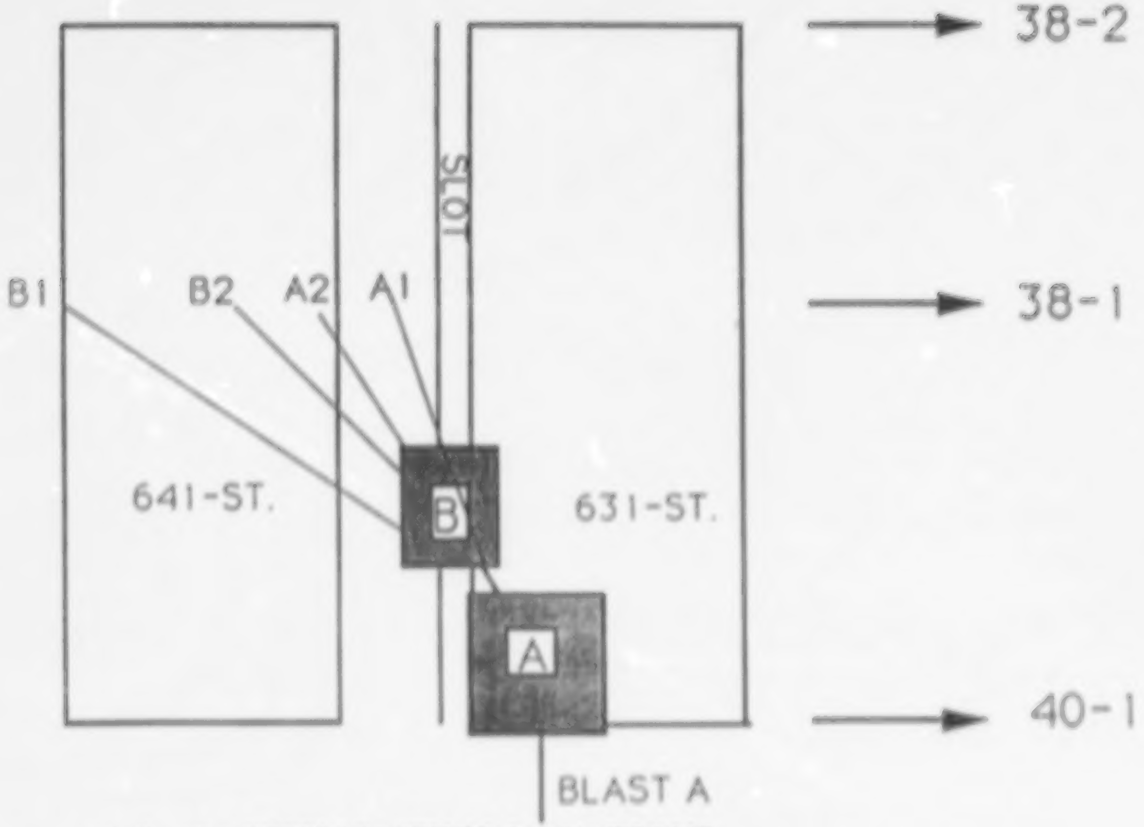

Blast location
Blast B
DIAMOND DRILL LAYOUT
1:250

Figure 7.17 Level plan showing location of blast monitoring instruments.


38-2
SLOT
B1
B2
A2
A1
38-1
641-ST.
B
631-ST.
A
40-1
BLAST A

A1 & A2 : SEISMOMETER LOCATIONS
FOR BLAST A, IN ROCK AND FIILL RESPECTIVELY

B1& B2 : SEISMOMETER LOCATIONS
FOR BLAST B, IN ROCK AND FILL RESPECTIVELY

Figure 7.18 Longitudinal section of 40-631 stope showing monitoring locations.

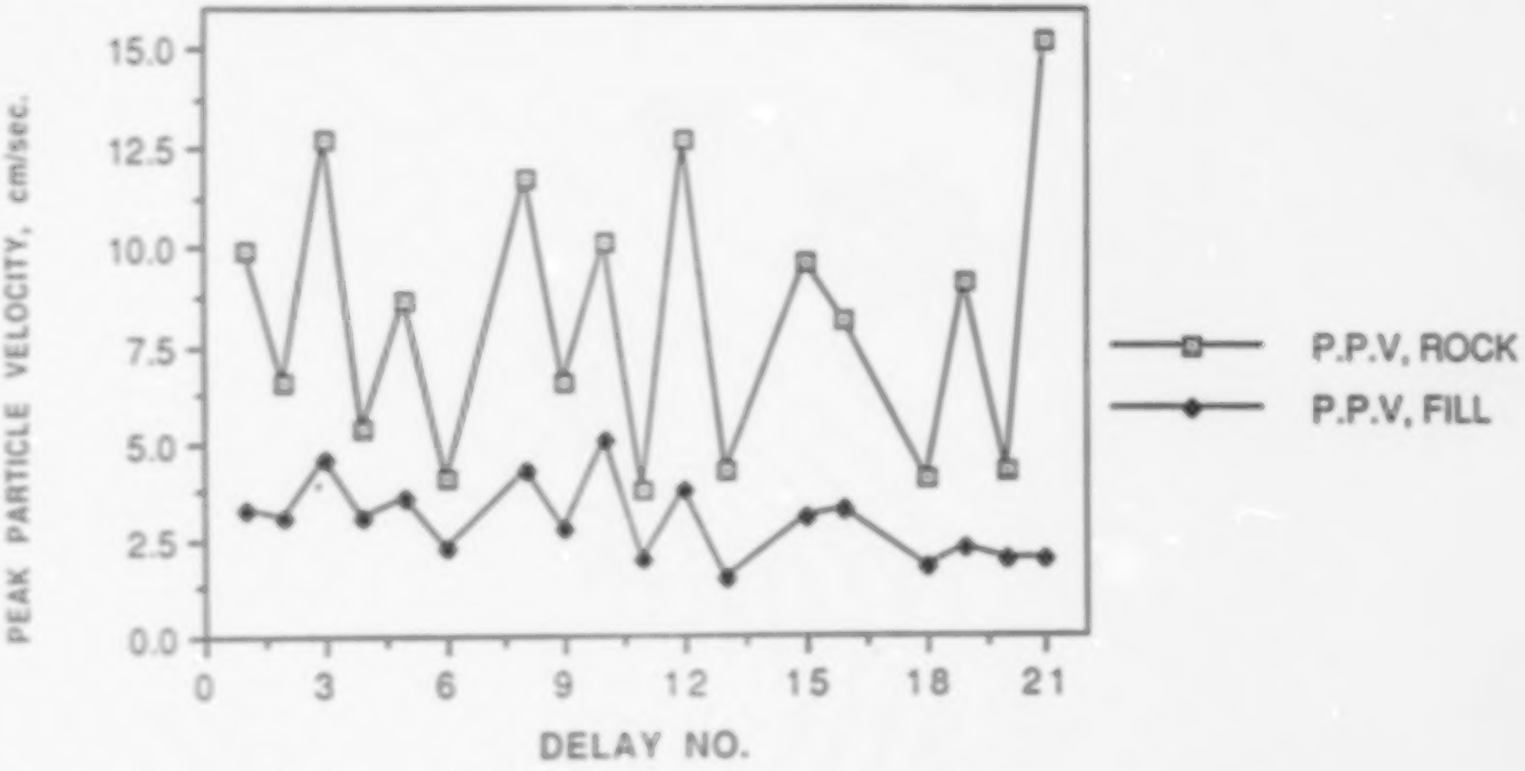


Figure 7.19 Seismic measurements in rock and fill from blast A.

CASE 4: In-Situ Rock Fill Testing Using a Pressuremeter: Due to the highly heterogeneous nature of rockfill masses, in-situ testing is needed to obtain the different physical and mechanical properties within placed backfill materials. In-situ testing of rockfill was mostly carried out in the 12-2-35 test drift at Kidd Creek Mine. A test drift 14 m long was driven in a stope which was filled earlier with consolidated rockfill. A longitudinal section of the drift is shown in Figure 7.20.

Pressuremeter Testing: The objective of this monitoring program was to evaluate the effectiveness of a pressuremeter testing technique for in-situ measurements of consolidated rockfill material. A TEXAM pressuremeter borehole device was used to run an in-situ loading test. The pressuremeter apparatus is made up of three components:

1. PROBE. The probe in the loading device which goes inside a borehole at the desired test elevation. The probe is essentially a hollow metallic cylinder on which an inner rubber membrane is fixed in the central part. An outer protective sheath is mounted over this rubber membrane and extends over the length of the probe. It is this sheath that is in direct contact with the walls of a borehole when the probe is pressurized.

2. CONTROL UNIT. The control unit is a fiberglass case with a front panel on which all the various regulators, pressure gauges, valves, etc. are fixed. Within the control unit is a reservoir which supplies water to a central measuring cell. Variations in volume during the test are read on a sight-tube.

3. COAXIAL TUBING. Two tubes connecting the control unit to the probe are arranged coaxially and are made of a semi-rigid material. The inner tube is used to apply water pressure to the central measuring cell, whereas the outer tube allows for the application of gas pressure to the guard cells. Expansion of the measuring tube when under pressure is negligible.

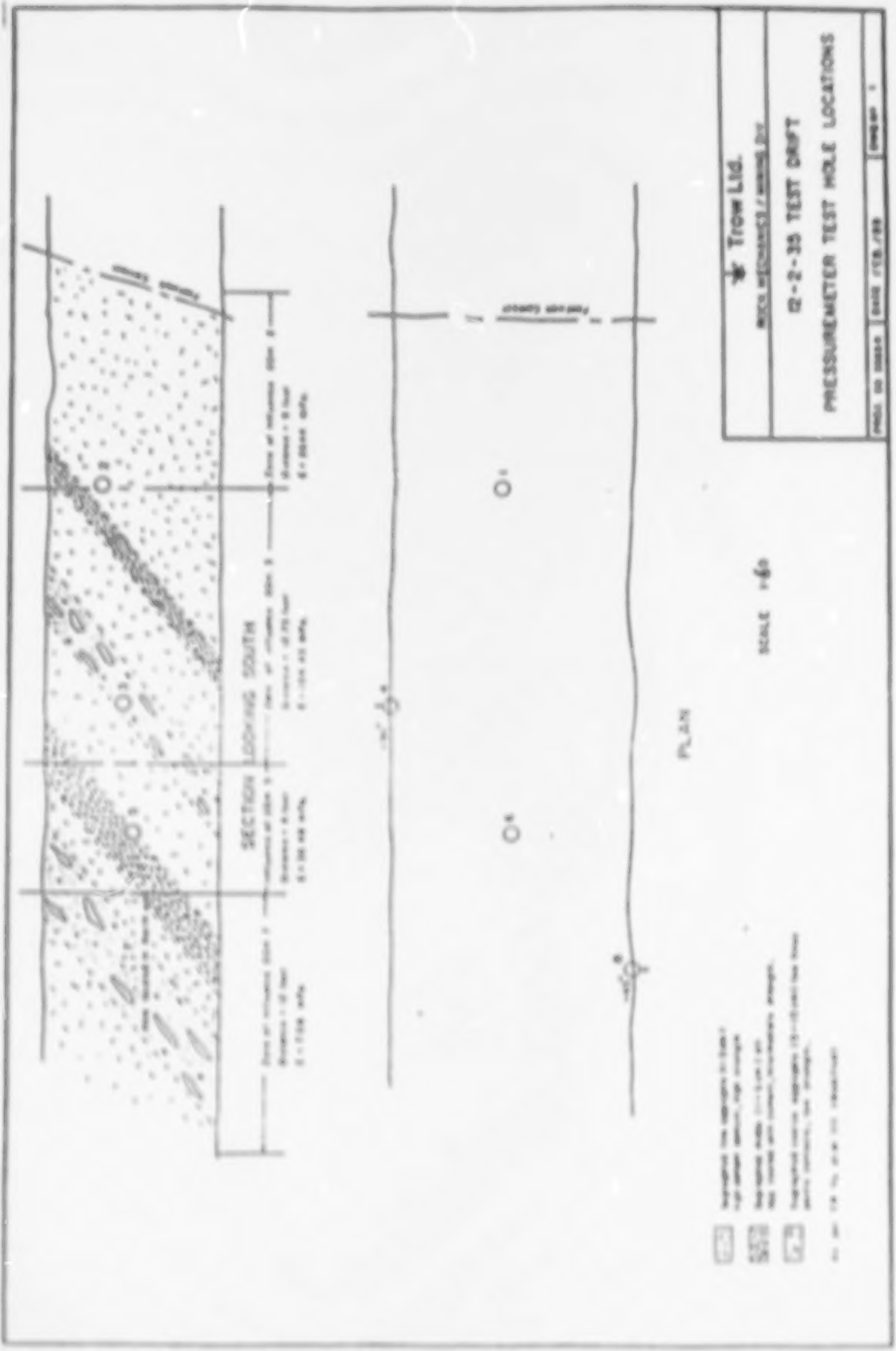

Trow Ltd.
12-2-35 TEST DRIFT
PRESSUREMETER TEST HOLE LOCATIONS
SECTION LOOKING SOUTH
PLAN
SCALE 1:60

Figure 7.20 Backfill test drift, pressuremeter test hole locations.

The pressuremeter test allows for the evaluation of rockfill quality at a specific depth and location in the filled stope. An in-situ stress / strain curve is obtained by plotting the injected volume versus pressure, as shown in Figure 7.21 The modulus of deformation, E, can be calculated from :

$$E = [(2 \times (1 + U) \times V \times dP) / dV]$$

where;
U = Poisson's ratio, estimated at 0.33
V = Volume of measuring cell
dP = Pressure increment
dV = Volume change

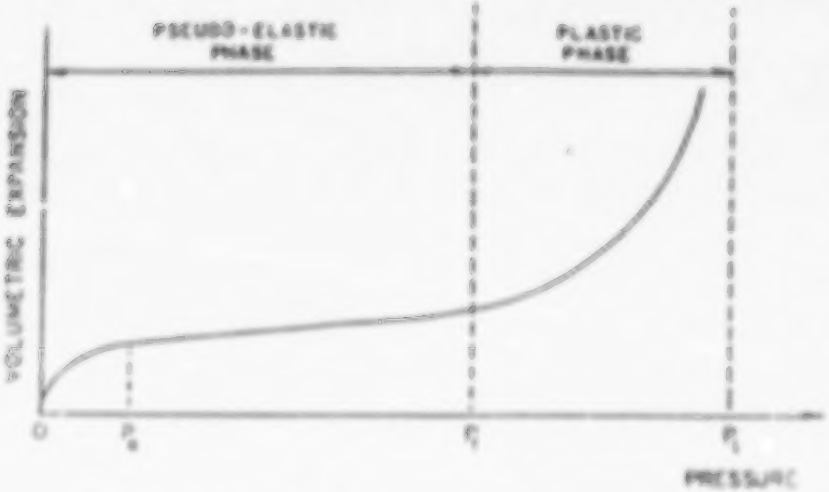


Figure 7.21 A typical pressuremeter test curve.

The loading device has to be set at the desired level of testing so that the materials to be tested undergo minimal disturbance. Once installed, the probe is submitted to an increasing pressure applied in equal increments. At each pressure stage, the volume changes of the probe are recorded at specific time intervals. The pressure / volume relationship is then drawn up for subsequent determination of material properties. Figure 7.21 shows a typical pressure / volume curve obtained from a pressuremeter test.

The typical pressure/ volume curve can be divided into three parts:

1. From $P=0$ to $P= P_O$: This portion of the curve corresponds to the seating of the probe against the wall of the borehole. The disturbance of the wall induced by drilling or driving the probe into place will have considerable influence on this portion of the curve. Also, the difference in diameter between the hole and the probe will affect the shape of the curve.

2. From $P=P_O$ to $P=P_f$: This represents the pseudo-elastic behavior of the loaded material. The probe is in contact with the walls of the borehole and the loading is uniform all along its length. The linearity of this portion of the curve helps define the modulus of deformation of the mass under load, which in turn can be used for settlement evaluation.

3. From $P=P_f$ to $P=P_l$: The pressure P_f by definition is the pressure at which the mass enters a plastic state. From this pressure on, the deformation of the mass under load accelerates up to a point where complete failure occurs. The pressure which defines failure is the limit pressure P_l.

For rockfill material, it is not always possible to get the pressuremeter probe into close contact with the sides of the borehole if the holes are uneven due to caving. Even if it is possible to insert the probe properly, sharp pieces of aggregate can puncture the sheath and membrane and cause the fluid to drain out of the probe. In the case of rockfill in-situ testing, the probe must be placed inside a special device which can provide the required support for the sides of the hole and protect the membrane. The device is a slotted tube which has six longitudinal, 2 mm wide slots cut through it. During testing, the slots open up and allow the tube to expand with the probe. The slotted tube has an outside diameter of 63.5 mm and an inside diameter of 47.5 mm and accommodates a standard AX probe of 44 mm. The slots are of the order of one meter length. All the above information & materials can be obtained from Roctest Limited, Montreal, Canada.

A total of eight holes were drilled using a drilling and casing technique developed for this project. The drilling was conducted using a JV drill on a jack-bar setup. Two holes, one horizontal and one vertical, were typically drilled from each setup to a nominal depth of 3 m. Drilling lines were tied into the footwall/backfill contact and were spaced at a spacing of 2.44 m. The probe was then placed at the test depth in each of eight boreholes. Precise locations are indicated on Figure 7.50.

Two to three tests were normally conducted in each borehole with the exception of borehole #7 which caved on retrieval of the rods. In total, 21 pressuremeter tests were conducted. Typical data relating to pressuremeter testing is presented in Appendix D and is summarized in Table 7.1.

The data indicated a general decrease in bulk modulus as holes moved away from the footwall. In fact, the only core which could be recovered was in the first drilling ring, closest to the footwall. The bulk modulus data was typically in the order of 1.5 GPa to 4 GPa in the first drilling ring, adjacent to the interface between rock and fill. This gradually decreased to a range between 100 MPa and 200 MPa in the second drilling ring as the ring moved toward the hanging wall, and was further reduced thereafter to the 7 MPa to 40 MPa range. This latter range indicated that the tested material contained little cement.

A second trend was also noted. With the exception of borehole #6, the down holes (1, 4, 5 and 8) indicated a greater variation in data than did the horizontal holes. This was to be expected as the down holes are more likely to traverse various fill layers than are the horizontal holes which were targeted to specific layers.

Core Sample Testing: To compare the results obtained from laboratory testing and in-situ borehole trials, the core recovered from borehole #2 was cut and trimmed to a height/width ratio of 2.5:1 and tested in an ELE-200 testing machine at a constant loading rate. A total of seven cores were tested. The individual stress/strain relationships are presented in Appendix D and the extracted data is summarized in Table 7.2. This phase of testing yielded a mean modulus value of 4.92 GPa and uniaxial compressive strength of 17.84 MPa. The mean modulus value obtained in the lab is in the order of twice that measured in the field. However, the lab data which related to the specific depths at which tests were conducted in the field correlate well with the field results. This is illustrated in Table 7.3.

Seismic Measurements: Two series of seismic measurements were carried out. The first series included field surveys and core sample measurements conducted by Trow Ontario Ltd. In the first field trial, the high attenuation of seismic waves in the 'porous' CRF and the low magnitude of the seismic source made the first arrival of seismic waves indistinct, giving in poor results.

To obtain seismic velocity, the second series of measurements were carried out by SIAL Geophysics Inc. using a OYO McSeis 160, 24 channel stacking digital seismograph. The determination of P-wave velocity was conducted in the field by using three shot points, two employing a hammer and one by blasting. Geophones were set up along a profile on the north wall and another one on the south wall of the test drift. The hammer shot points, SP-1 and SP-2, were at 5E on their respective lines and the blast was set off at the east end of the drift in CRF on the north wall. For more information refer to Appendix D.

From the hammer shot points, P-wave velocities of 3080 and 3610 m/s were obtained. At SP-4, closest to the blast, the P-wave velocity was rather low, probably because of nonrepresentative full body wave. The next interval 7.1 m away yielded a velocity of 2960 m/s. The average P-wave velocity in CRF from all three readings was 3220 m/s.

Laboratory seismic velocity measurements were conducted on core samples from borehole #2 using the PUNDIT instrument and the data is presented in Table 7.4. The average velocity of 3,080 m/s measured from core samples was in agreement with that of 3,220 m/s obtained from the field measurement by SIAL.

Plate Load Test: To develop a quantitative method for determining the in-situ strength of consolidated rockfill, a series of plate-load tests were carried out on backfill material in the backfill test drift. In the plate-load test, a steel plate 10 cm x 10 cm x 2.5 cm thick, was used as a bearing plate and load was applied on it with a 30 tonne capacity ram until the fill under the plate failed. Tests were conducted in chambers which were carefully prepared in the fill to be tested. Testing on the exposed fill (drift wall) was accomplished by holding the ram against a timber post to the opposite drift wall while loading of the ram was in progress.

The test results indicated that the bearing plate-load test would provide a quick and reliable method for determining the in-situ strength of consolidated rockfill, provided test sites can be made readily available. The ratio of ultimate bearing strength to uniaxial strength was found to be approximately 3.2 to 1.

Borehole #	Depth (m)	Bulk Modulus (MPa)
1	0.92	>4000
	2.14	1470
2	0.92	3383
	1.83	2270
	2.75	2280
3	0.92	96.0
	1.83	192.2
	2.14	144.3
	2.75	185.3
4	0.92	37.98
	1.83	2.19
	2.75	30.18
5	0.92	6.31
	2.14	44.87
	2.75	28.10
6	0.92	28.20
	2.14	12.87
	2.75	7.91
7	2.14	7.06
8	0.92	22.01
	2.14	8.05

Table 7.1, Summary of pressuremeter tests.

Sample Depth (m)	Deformation Modulus (GPa)	UCS (MPa)
0.61	4.31	19.0
1.22	3.71	10.0
1.83	6.45	17.0
2.14	3.40	21.5
2.44	6.7	17.0
2.90	4.10	20.0
3.05	5.75	20.0
Average	4.92	17.84
StandardDeviation	1.25	3.59

Table 7.2, Summary of laboratory testing

FieldData		LaboratoryData	
Depth (m)	Modulus (GPa)	Depth (m)	Modulus (GPa)
0.92	3.38	0.61	4.31
0.92	3.38	1.22	3.71
1.83	2.27	1.83	6.45
1.83	2.27	2.14	3.40
2.75	2.28	2.90	4.10

Table 7.3, Comparison of field and laboratory data.

Sample Depth (ft)	Distance (in)	Time (μs)	Velocity (ft/s)
2	9.252	73.6	10 475
3	6.130	46.3	11 033
4	15.729	121	10 832
6	5.857	53.7	9 089
7	14.536	115	10 533
9.5	5.799	50.6	9 550
10	11.625	105	9 226
Average			10 106

Table 7.4, Summary of P-wave velocity measurements, Laboratory trials (PUNDIT)

CASE 5: In-Situ Hydraulic Fill Testing (1989): As part of a previous Falconbridge, CANMET joint project, "In Situ Monitoring and Computer Modeling of a Cemented Sill Mat and Confines during a Tertiary Stage Pillar Recovery", (DSS File No. 06sq.23440-5-9205), a series of in situ geotechnical investigations of consolidated hydraulic fill were conducted. Trow Ontario, consulting engineers were retained as principal subcontractors to carry out part of the actual work. The investigations were conducted at Falconbridge's Lockerby mine in Sudbury, Ontario. This case study is a short review of this extensive testing program.

The purpose of the testing program was to determine in situ physical property data and monitor behavior of hydraulic fill adjacent to mining operations. The information obtained was used as input to a computer model of fill behavior in a sill mat. The following instrumentation was installed;

1. Geophones for blast vibration monitoring.
2. Gloetzl cells for measurement of fill loading.
3. Soil extensometers for fill displacement measurement and failure profile determination.
4. USBM ultra sonic measuring device for fill failure profile determination.
5. Texam pressuremeter for fill elasticity determinations.
6. Penetration cone for fill strength determination

A primary goal of the project was to develop a drilling technique which would allow the recovery of undisturbed backfill samples and produce an accurate drill hole for instrumentation and monitoring. Previous to this project, recovery of fill samples using conventional diamond drilling had proved to be unacceptable, average 15% or less. The project was particularly successful in this regard; drill core recoveries of 95% being achieved.

Development of Drilling Technique: Previous field experience with drilling in hydraulic fill and recommendations from drill contracting firms familiar with this type of drilling indicated that the following drill performance criteria were required;

1. Accurate control of drilling rotation in the 50 to 200 rpm range.

2. Minimal sample contact with drilling fluids.

3. Drill barrels that allow for easy core removal.

To meet these requirements the following drill was specified;

Diamec 251 (modified)
Diamec 260 low rpm/high torque rotation unit
Diamec 260 rod holder
Skid type frame
30 kW 550V electric motor
Craelius T6s 86x1500mm core barrel, (double tube, swivel head type)
Split inner tube core barrel
Surface set diamond drill bit - 20 spc

The drill operating parameters were set as;

Feed load 0.5 to 0.75 tonne
Water pressure 200 to 400 kPa
Feed rates 15 to 25mm/min
Rotation 50 to 120 rpm

These were varied by the operators as conditions allowed in order to provide optimum penetration with minimum "flushing" requirements. A primary concern in the recovery of fill core is the minimum influence of drilling fluid on the core. The flushing agents used in drilling are important because they can affect the moisture content and physical integrity of the core samples. A range of flushing techniques were evaluated; pressurized air, pressurized water and co-polymer mixed with pressurized water. The final flushing agent used during drilling operations was a concentrated co-polymer of sodium acrylamides/acrylates as supplied by Westcoast Drilling Supplies Limited. This solution worked well and core recoveries were good even in weaker cemented fill.

The careful handling of fill core was essential to the good recoveries achieved. Both inner split tube and standard NW solid tube (both using double tube core barrels) proved effective. The split barrel design provided easier logging and handling of recovered samples.

The drilling and sampling technique developed in this project proved capable of core recoveries in excess of 90%, a excellent performance in weakly cemented hydraulic fill. Figures 7.22 and 7.23 indicate the location of the drill holes.

Trial Continuous Flight Augers: Backfill sampling of hydraulic fill with a continuous flight auger was also tested in this project and proved be economical and accurate method of drilling holes for placement of instrumentation.

Instrumentation: A short summary of the instrumentation results are presented here, detailed information can be obtained from the original project report available from CANMET. Table 7.? gives a summary of the instrumentation.

Instrumentation	No. Units	Installed By	Purpose
Geophones	4	Falconbridge	PPV Blast Wave Frequency
Gloetzl Cells	4	Falconbridge	Fill Loading
Soil Extensometers	3	Falconbridge	Fill Failure Profile
USBM Ultra Sonic Measuring Device	1	Falconbridge	Fill Failure Profile
Texam Pressuremeter	5	Trow Limited	Modulus of Elasticity
Penetration Cone		Trow Limited	Fill Strength

Table 7.5 Summary of Instrumentation.

Results: Gloetzl pressure cell measurements indicate that a negligible amount of fill loading occurred after the adjacent stope was mined out. This supports the premise that, because the rock is much stiffer than the fill, significant load transfer does not occur until the fill has undergone considerable compaction, (something in the order of 15% wall convergence).

Pressuremeter results may be summarized as follows;

	Strongly Cemented Tailing:Binder < 10	Weakly Cemented Tailing:Binder > 25
Pressuremeter modulus	400 - 600 MPa	40 - 200 MPa
Pressuremeter limit pressure	10 - 20 MPa	5 - 15 MPa

Extensometer and USBM ultrasonic measuring device results were used to map the outline of the fill failure which occurred during mining and was used to calibrate a numerical model.

Blast Vibration Monitoring: The first ten blasts in 31-176 VCR stope were monitored and the following results obtained and subsequently used in numerical modelling;

PPV*	53 cm/sec
Dominant Frequency	30 Hz
Blast Duration**	15 ms

* Measured 15m from the blast

** Blast duration for the hole adjacent to the fill wall.

Figures 7.24 to 7.27 detail the blast vibration results and monitoring locations.

Conclusions: The development of a reliable drilling technique for the recovery of weakly consolidated hydraulic fill was a significant development in this case study. The material properties obtained by the instrumentation were used as input to a numerical model which was calibrated against an observed fill failure.

The work reported in this case study was not conducted as part of this project so details have not been included as an appendix. Refer to CANMET for a copy of the original report and detailed results.

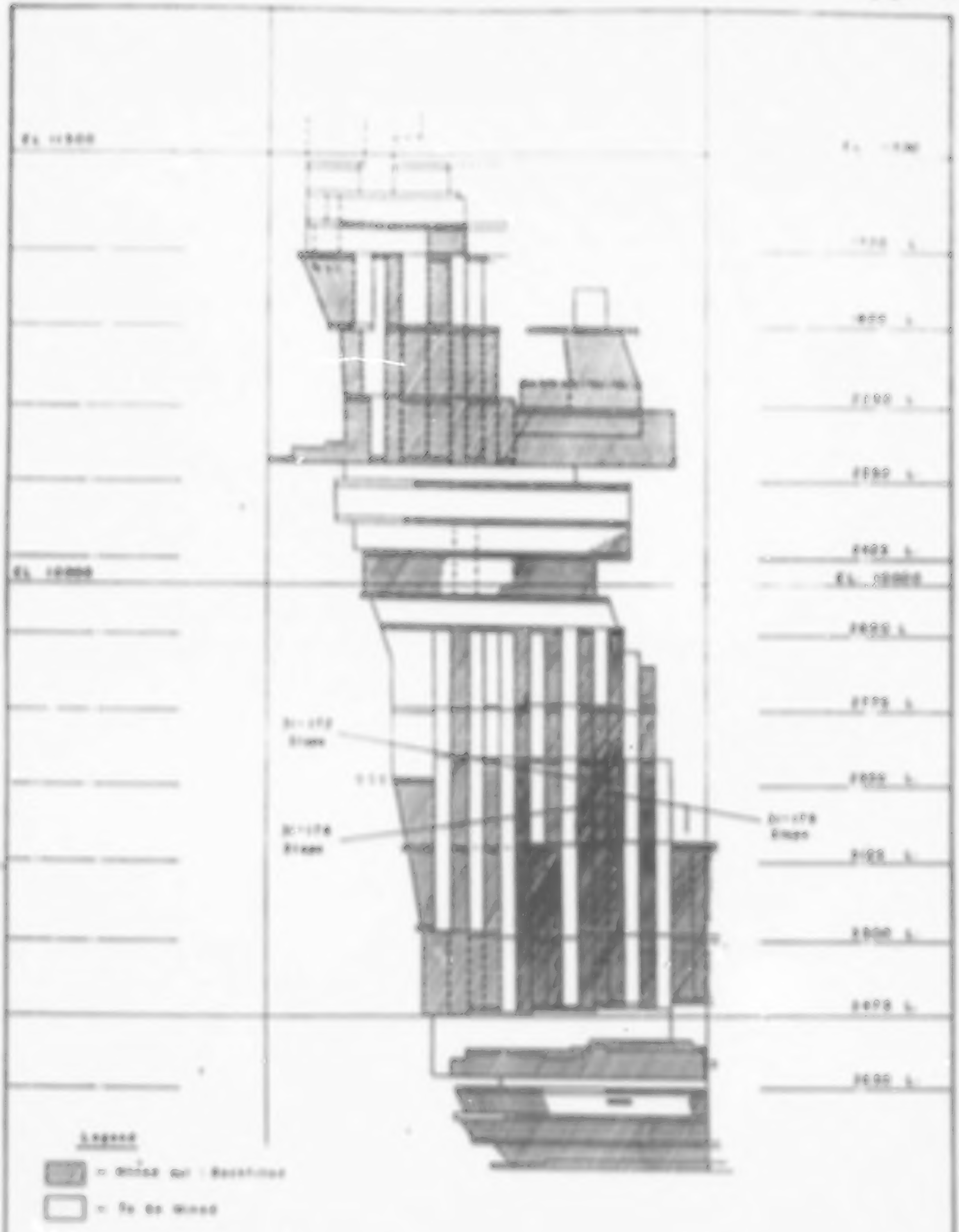

EL 10000
EL 10000
Legend
= Mined and Backfilled
= To be Mined

Figure 7.22 Section through Lockerby mine showing location of test stopes.

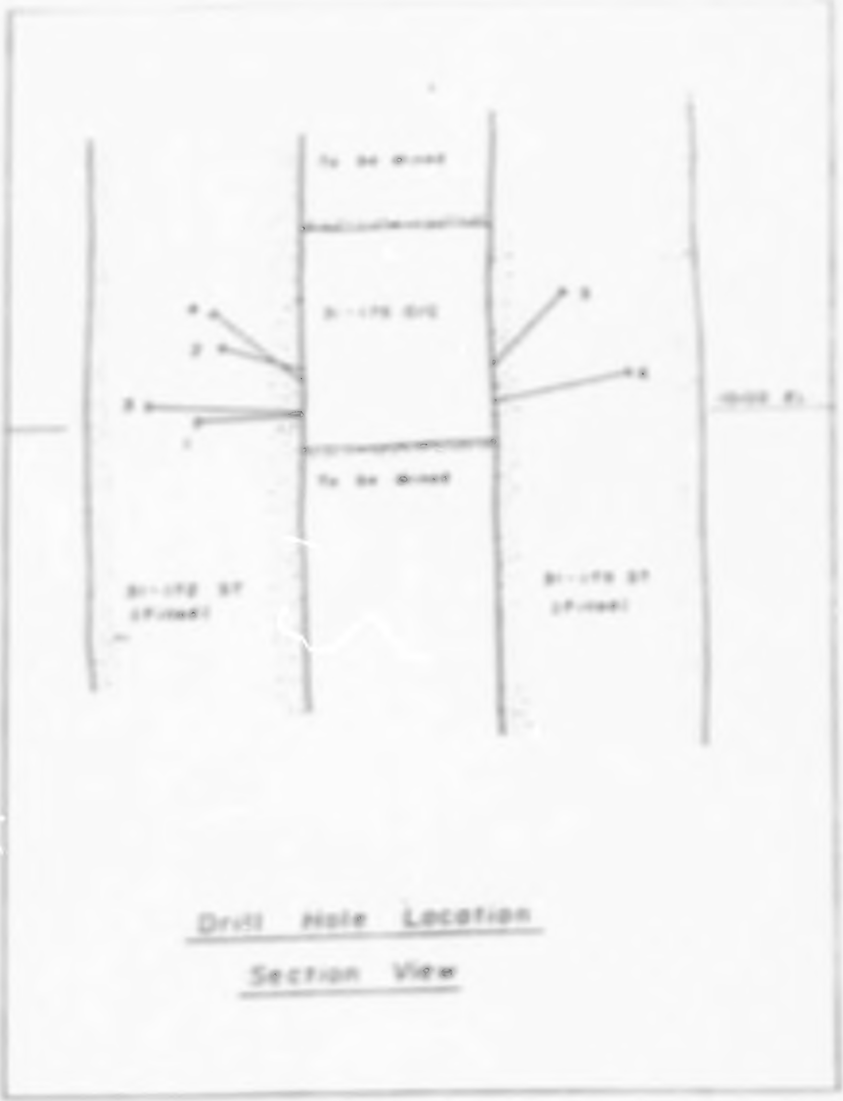

To be mined
To be mined
(Filled)
(Filled)
Drill Hole Location
Section View

Figure 7.23 Location of fill holes.

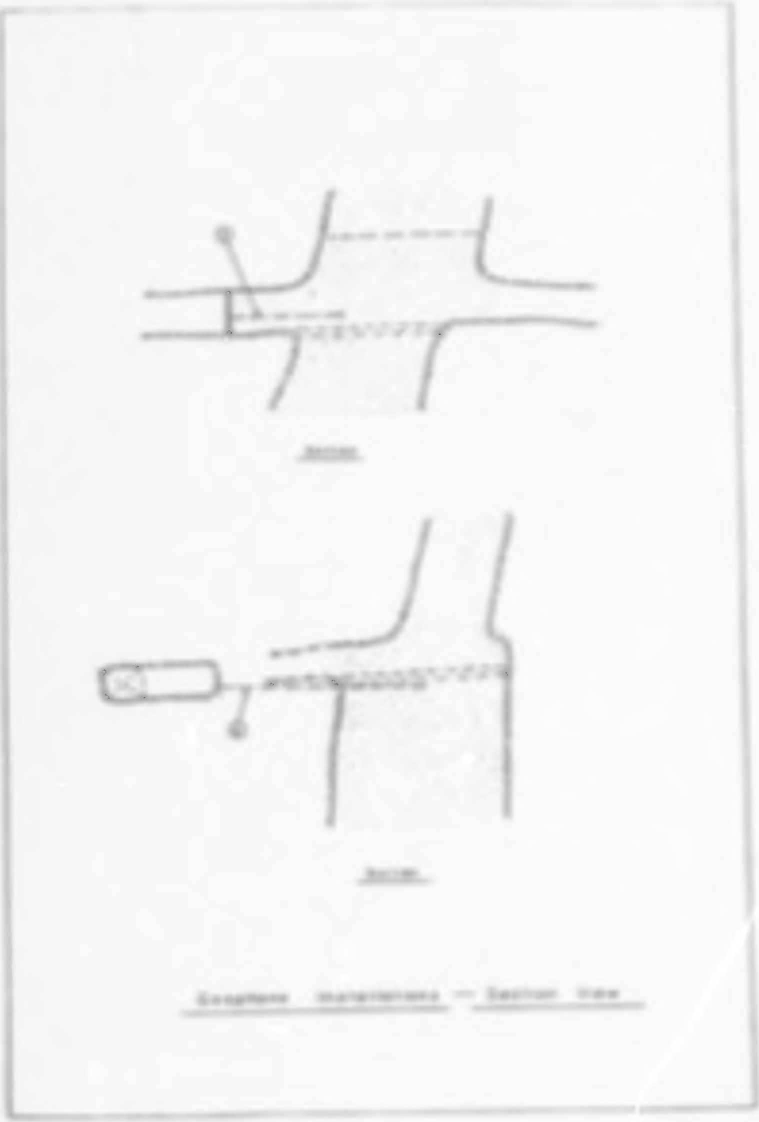

Section
Section
Geophone installations — Section View

Figure 7.24 Geophone location for blast vibration monitoring.

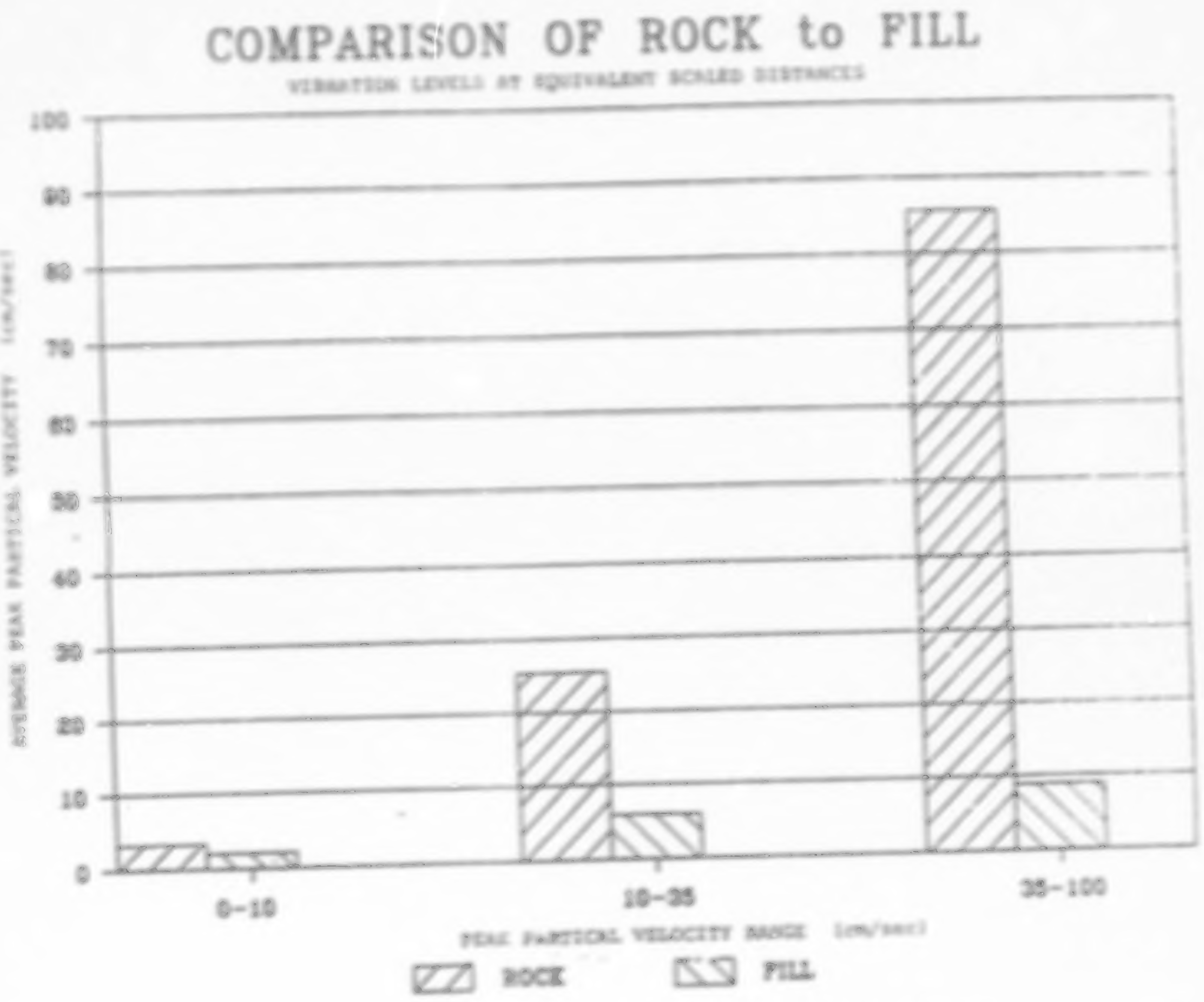


Figure 7.25 Results of blast vibration monitoring.

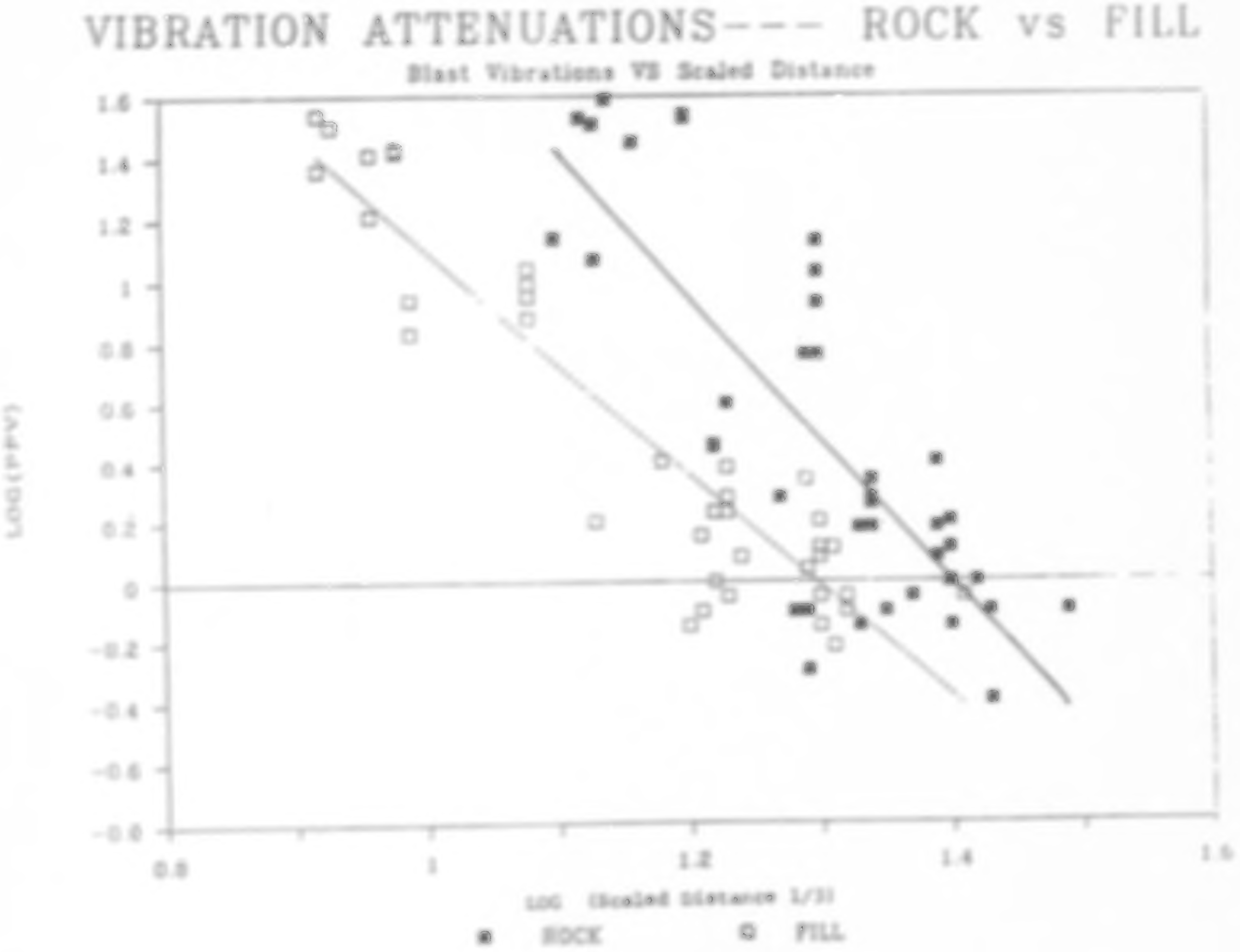


Figure 7.26 Blast vibration results.

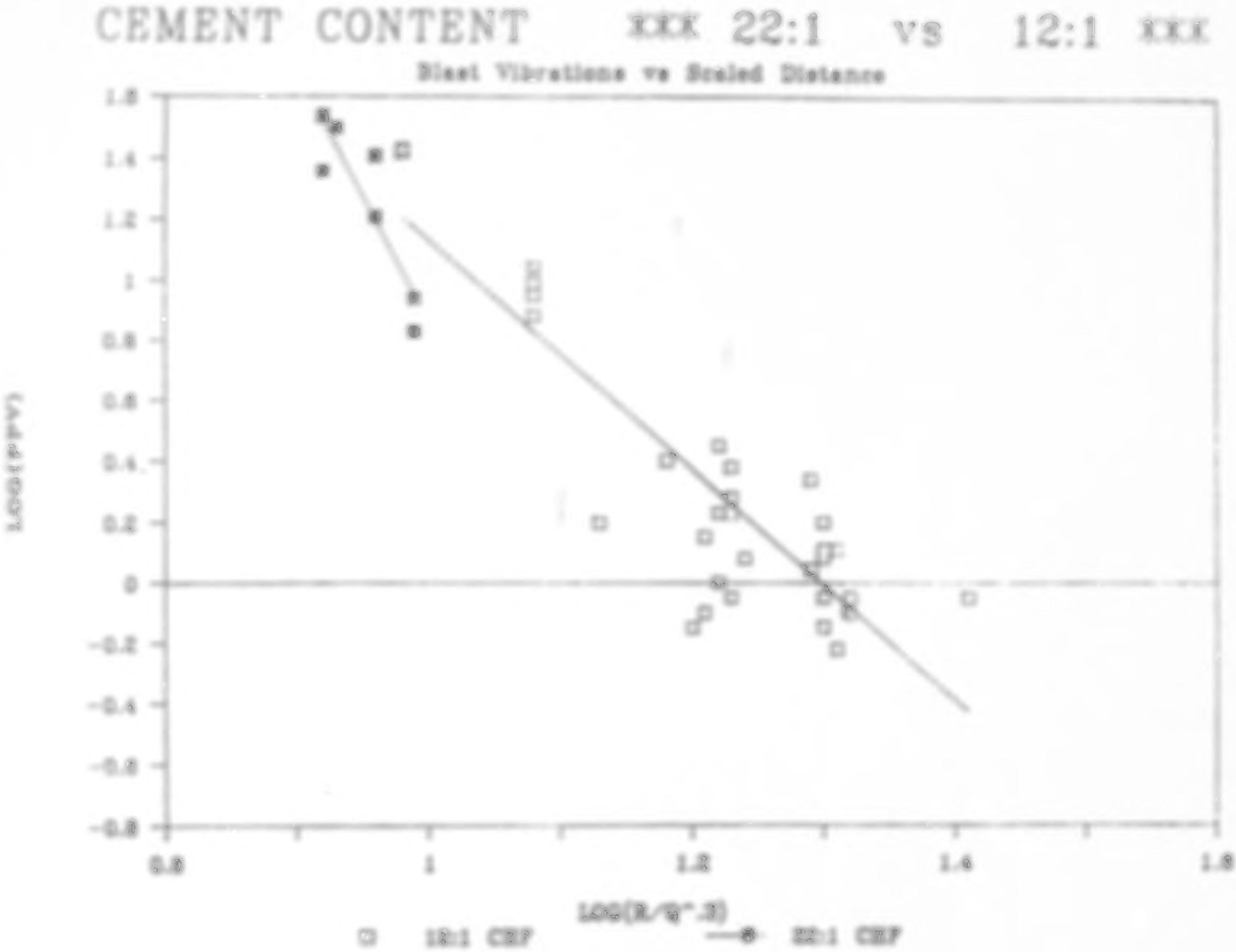


Figure 7.27 Blast vibration results.

CASE 6: Monitoring of Pressures in Hydraulic Fill During and After the Filling of Blasthole Stopes, Application of Gloetzl Cells: In order to ensure that the pressure acting against fill barricades at the base of blast hole stopes being hydraulically filled was not exceeding 100kPa, (as specified by the Occupational Health and Safety Act of Ontario) a series of pressure readings were taken at the base of several stopes. The work was conducted at Falconbridge's Strathcona mine, (reference in an internal Falconbridge report, unpublished, Udd, 1984).

The practice at Falconbridge mines is to pour a plug of consolidated fill at the base of completed blasthole stopes as containment for the bulk fill poured above. In order to comply with the regulations is was necessary to determine what pressures were being appliedto the containment plug and how much was being transmitted to the light fill fence (chapter 4 describes the design of fill containment plugs).

Prior experience with the use of simple pressure gauges installed through fill fences, used to monitor fill pressures had not been very successful. As a general rule the gauges were fitted to the ends of perforated pipes which passed either into or through the consolidated containment plug. Experience with gauges installed in the plug (just inside the fill fence) was usually that a small pressure was developed as the pipe was buried in fill. With the curing of the plug and drainage the pressures usually decreased to zero. Further filling of the stope was not seen to have any effect on the hydraulic pressures in the plug of cemented fill. Gauges installed on pipe which extended through the plug to the base of the bulk fill usually showed increases in pressure as filling progressed, as would be expected.

It was often difficult to interpret the readings from the gauges attached to pipes passing through the plug because water trapped in the capped and instrumented lines could not escape. The gauge would read a permanent pressure due to this head of water which did not reflect actual pressures in the fill. It was necessary correct readings to account for this potential false head. The sensitivity of such instrumentation was poor and the accuracy questionable. In addition the presence of fine particles in the fill water caused the gauges to become plugged and stop functioning. Others could not be read because of opaque coatings (calcium carbonate from cement) obscured the dials.

In order to improve the validity of pressure readings it was decided that Gloetzl type earth pressure cells should be tested. The Gloetzl cell consists of two thin plates welded together and filled with a fluid. The cell acts as a bypass valve in a hydraulic circuit. A pump is used to supply fluid to the cell until the internal pressure becomes equal to that which is applied externally; at which point the bypass valve opens. A precision hydraulic gauge is used to determine the point at which pressure equalization takes place. The pressure is measured in only one dimension, perpendicular to the plate surface. The advantage of these cells is that they may be installed at a remote location, the only requirement being the provision of a connection between the cell and the pump via double tubed hydraulic line. The pump and gauge for reading cell pressures is portable, although rather heavy to carry around.

The performance of the Gloetzl cells was excellent. No cells failed nor were any of the readings rejected. The long term stabilities of the cells have been good, two cells were still functioning well some eight months after installation. It is important to note that the cells were installed so that the hydraulic lines lead downwards from the cell to, and through, the fill plug. This was done in order to avoid the higher internal pressures that would have resulted in the cells had the lines been directed downwards from the overcuts. Such pressures would have caused higher initial readings, some ambiguities, and restrictions on the effective range of the cells.

At initial installation a base or "zero" reading must be taken on a Gloetzl cell before any load is applied. Actual values of pressures resulting from loading are calculated by subtracting the base reading from subsequent values. The base pressure is a consequence of pre-pressurization of the cell during manufacturing and the difference in elevation between cell and gauge.

Results: The common results of the monitoring program are summarized as;

1. The placement of fill on top of a cell results in increased pressure. The magnitude of the increase was strongly influenced by the permeability of the fill; in less permeable strongly cemented fill the effects of even substantial additions of fill above was small, in more permeable fill, less cemented fill, the effects were much more rapid and significant.

2. The rates of decrease in pressure are greatest in the more permeable fills, and the decrease continued for several months.

3. Variations in the rate at which fill is added are reflected in the subsequent pressure changes.

4. Pressures observed in the fill containment plugs were very small while the filling of the panels above took place. This indicates that pressures acting on the light fill fences were also very small.

5. The pressures recorded against the top of the fill plugs were as expected. The maximum pressure measured was 200kPa even though the depth of fill above was 32 - 35m. This is much less than the theoretical potential pressures. The fill was substantially self supporting. The pressures measured declined as a result of drainage for several months.

Table 7.6 is a summary of stopes monitored.

Mine and Stope	Dimensions (m)	Fill Plug Ratio	Bulk Pour Ratio	Fill Fence Monitoring Max Press. (kPa)	Fill Fence Monitoring % of Possible	Bulkhead Monitoring Max Press. (kPa)	Bulkhead Monitoring % of Possible
Fraser 34-0-564	H = 19.5 L = 37.9 W = 10.0	16:1	32:1	3.5	1	103	41
Strathcona 27-05-10	H = 36.9	16:1	32:1	62	33	124	30
Strathcona 27-05-10	H = 36.9	16:1	32:1	-	-	41	7.5
Strathcona 27-C8-C9	H = 55.4	16:1	32:1	-	-	21	3.5
Strathcona 27-18-D7	H = 51.8	8:1	32:1	-	-	159	20
East 5808-149-160	H = 51.2	30:1	30:1	10	-	-	-

Table 7.6, A sample of blasthole stope fill fence and fill plug pressures measured with Gloetzl cells.

Conclusions: The pressure readings indicated that pressures within fill containment plugs are very low, that pressures acting on the plug from above are moderate so long as drainage is adequate, and that the fill plug is a safe containment system.

The work reported in this case study was not conducted as part of this project so detailed results have not been included as an appendix.

CASE 7: Experience With In Situ Testing of Consolidated Rockfill: Pressuremeter and dilatometer testing of in situ rockfill was carried out at Falconbridge's Strathcona mine in order to determine the fill stiffness, (internal Falconbridge report by P. Hopkins 1988, "Strathcona Deep Copper Consolidated Rockfill Option, Background Data", not published)

In situ testing of various types of rockfill was conducted to correlate laboratory measured strength and stiffness parameters to in situ values. Rockfill quality is very sensitive to placement method and conditions. These conditions are not easy to simulate in the laboratory, and so it is important to correlate laboratory strengths to those actually achieved in place. Pressuremeter and dilatometer instruments were used to measure the in situ strengths. Recovered core was tested in the laboratory and compared to control cylinders cast in the laboratory. Trow Ontario Ltd were responsible for much of the test work described in this case study.

Two types of rockfill were tested in this program;

1. Waste rock dumped in place and a cement grout then pumped over the top. The cement was intended to percolate through the rock to form a competent consolidated rockfill. In practice it resulted in a poor quality fill because the grout tended to follow drainage paths rather than uniformly percolating through the whole mass.

2. A bucket mixed rockfill, where grout was spayed on waste in the bucket of a loader, the dumping and pushing action mixed the rock and grout to give a reasonable coating of cement on the rock.

Drilling problems: Significant problems were experienced when trying to drill in poorly cemented rockfill (mostly the rockfill which had cement grout percolated through). Three inch diameter holes were cored in the fill, but they all caved and could not be used except in the upper better grouted portion.

A system of pre-installed split tube testing sleeves was adopted for subsequent testing in this material. Split PVC casing pipes were installed in place prior to backfilling. The pipes were infilled with various materials to give them internal stability during the filling operation. Rock salt was found to be the most suitable fill material, as it could easily be washed out when the hole was required for testing. Pre installed pipes proved to be a successful method of providing access for in situ testing in poorly cemented rockfill.

Pressuremeter Tests: Two sets of pressuremeter tests were conducted; those in split casing holes and those in drilled holes. Substantial data scatter was observed. In some instances the data from two adjacent tests within a given hole varied by an order of magnitude. The following reasons were proposed;

1. The in situ rockfill is composed of poorly cemented zones.

2. Higher results can be obtained due to eccentric loading resulting from large continuous blocks adjacent or close to the hole wall.

3. Lower values can be obtained if a large portion of the contact area between probe and hole wall is composed of voids.

It is recommended that in future work with this type [illegible]erial a borehole camera be used to examine the surface conditions in the hole.

The parallel series of pressuremeter tests in cased and uncased holes yielded results of the same order of magnitude. The values obtained from cased holes was typically 2 to 3.5 times higher than those obtained in adjacent drill holes. This is probably due to the drilling induced damage to surrounding rockfill in drilled holes and because portland cement paste preferentially segregated out of the rock paste mixture along the split casing. This would result in higher values due to concentration of binder close to the casing.

Laboratory Testing: Laboratory testing of uniaxial strength and stiffness were conducted on both cored samples and laboratory cast cylinders. There was good correlation between the cored and the cast samples in both strength and stiffness. A comparison of laboratory and field stiffness modulus however, showed a significant variance. The comparison indicated that the laboratory values were between 10 and 120 times greater than field observed stiffness values. These differences are probably due to size effects.

Tables 7.7 to 7.9 give the results obtained in this testing program.

Pressuremeter Test Summary

Fill Area E-3 3900 L - Strathcona Mine

Hole Number		Depth of Hole (m)	Test Depth (m)	Modulus (MPa)	Comments
Drilled	Cased				
	1	1.10	.76	14.26	Shallow hole, one test performed, casing broke
1		0.91	.30	69.06	Probe membrane burst
1			.61	6.43	
	2	1.22	--	---	Casing bent, testing not possible
2		1.07	.30	219.36	Probe membrane burst
2			.61	17.44	
	3	1.22	.30	130.98	Probe membrane burst
	3		.91	23.81	
3		0.91	--	---	Large void at .61 m in test zone, no test performed
	4	1.52	.45	78.42	
	4		1.22	83.32	
4		0.91	--	---	Large void .53 to .76 metres, no test performed
	5		.30	---	
	5	1.22	.91	46.75	
5		1.22	.30	13.52	Probe membrane burst
5			.91	14.28	
	6	1.22	.30	17.22	
	6		.91	48.49	Casing broke
6		1.22	.30	5.74	
6			.91	10.39	
	7		.30	90.45	
	7		.91	65.53	
7		0.69	.30	139.62	
	8	1.22	.30	100.01	Casing broke
	8		.91	34.42	
8		.99	.30	15.77	
8			.61	26.0	
	9	1.22	.45	54.60	Probe membrane burst
	9		.91	77.11	
9		0.76	.45	15.83	
	10		.45	54.90	
	10	1.22	.91	73.31	
10		0.91	.61	31.4	Void at .30

Table 7.7 Summary of pressuremeter test results.

Laboratory to Field Comparisons

Mix	Cast		Cored		Cased	Drilled
#	E (Gpa)	c (MPa)	E (Gpa)	c (MPa)	E (MPa)	E (MPa)
1	2.53	3.67	3.62	9.51	14.26	69.06 6.43
2	3.05	4.00	2.20	10.64	--	219.36 17.44
3	5.04	6.92	2.11	10.45	130.98 23.81	--
4	2.08	4.24	--	--	78.42 83.32	--
5	3.03	4.95	5.91	12.1	46.75	13.52 14.28
6	3.52	6.67	1.81	7.28	17.22 48.49	5.74 10.39
7	5.75	7.76	--	--	90.45 65.53	139.62
8	2.74	4.81	2.67	6.71	100.01 34.43	15.77 26.0
9	3.64	4.93	4.11	13.48	54.60 77.11	15.83
10	2.69	4.73	2.40	5.77	54.90 73.31	31.4

Table 7.8 Comparison of strength and stiffness between cast and cored rockfill samples.

table 7.9

Comparison of Cast and Cored Sample Results

Grout Density	NPC Addition Rate (%)	Cast Samples			Cored Samples		
		Sample #	E (Gpa)	σc (MPa)	Sample #	E (Gpa)	σc (MPa)
50:50	3	1	2.62	3.76	1a	3.12	10.11
		1b	2.43	3.58	1b	4.12	8.90
	5	2	1.95	4.01	2a	--	--
		2b	4.15	3.99	2b	2.20	10.64
	7	3	5.33	6.12	3a	1.91	13.85
		3b	4.74	7.2	3b	2.31	7.05
	8	4	2.53	4.36	4a	--	--
		4b	1.62	4.12	4b	--	--
	10	5	2.94	4.24	5a	9.80	17.52
		5b	3.11	5.66	5b	2.31	6.76
60:40	3	6	3.34	5.10	6a	--	--
		6b	3.70	8.24	6b	1.81	7.28
	5	7	5.87	8.67	7a	--	--
		7b	5.62	6.85	7b	--	--
	7	8	1.64	3.08	8a	--	--
		8b	3.83	6.53	8b	2.67	6.71
	8	9	4.97	7.35	9a	4.11	13.48
		9b	2.31	2.51	9b	--	--
	10	10	2.71	4.71	10a	--	--
		10b	2.67	4.74	10b	2.40	5.77

Table 7.9 Comparison of laboratory against field results.

Conclusions: The scatter in data from these tests may be attributed, in part, to size effects. The pressuremeter will have a volume of influence considerably larger than the volume of a test cylinder. Typically, as the volume of a sample increases, the material properties (strength) decrease. This problem with size effects is common through out all testing in rockfill. The size of the particles makes it difficult and expensive to work with samples of sufficient size to get a representative estimate of material properties.

The pressuremeter is considered an applicable method for testing rockfill. It is however subject to equipment and test procedure problems, and limitations in terms of working pressures and stiffness. The pressuremeter is prone to membrane failure in rockfill materials.

Bucket mixed rockfill gave a much superior product to the grout percolated fill. The grout failed to give a uniform coverage of the rock and resulted in parts of the fill remaining completely uncemented.

The work described in this case study was not conducted for this project and so the detailed results have not been included as an appendix.

CASE 8: In Situ Hydraulic Fill Instrumentation (1984): An in situ backfill trial was undertaken at Falconbridge's Lockerby mine during recovery of the first blasthole stope (22-139) adjacent to a 12:1 consolidated hydraulic fill pillar, (internal Falconbridge report by D. Dileo, 1984, unpublished). The purpose of the trial was to record rock and fill response to mining with the ultimate aim of assessing whether reductions in the binder contents of future fill pours were feasible.

As the 22-139 stope was being mined, monitoring of the ground surrounding the stope was carried out through;

1. Dilatometer testing of the fill pillar (23-143 stope) at the overcut and undercut elevations to assess the impact of mining on the deformation properties of the fill.

2. Extensometer monitoring in the hanging wall and the ore and fill pillars immediately adjacent to the 22-139 stope to determine the magnitude of ground movements occurring during the different mining stages.

3. Visual inspections and profiling of the fill pillar wall exposed during mining to assess the impact of blasting on dilution originating from the fill pillar and to gain information about fill pillar structure.

4. Limited vibration monitoring of backfill.

Dilatometer Testing: Dilatometer testing was carried out by CANMET personnel in 5 holes drilled into the fill pillar at the overcut and undercut elevations of the 22-139 stope, (detailed reference Swan & Vaillancourt, 1983). The testing was done on a weekly basis beginning one week prior to production blasting in th 22-139 stope. Measurements were subsequently taken after each successive blast to establish whether changes were occurring in the deformation properties of the fill as mining progressed. The conclusions from this work are;

1. Measured values of the in situ deformation modulus of the fill varied between 0.9 GPa and 1.2 GPa. The dilatometer and laboratory modulus results were in the same order of magnitude but the dilatometer results were consistently about 2 to 3 times lower than the laboratory modulus measured on recovered core samples. This may be due to size effects; the dilatometer is measuring the modulus over a much larger volume of fill material than the core samples, and material properties tend to deteriorate with sample size.

2. The deformation modulus values measured increased as mining progressed.

Extensometer Monitoring: Three extensometers were installed at the locations shown in figures 7.28 to 7.30.

Hanging wall movement was measured by extensometer 20-114, installed in a downhole from the overcut elevation. As expected, hanging wall movement towards the open stope increased as mining progressed with the greatest movement occurring at the anchor closest to the ore contact. The maximum movement, (0.26m) was measured at approximately the mid-height of the hanging wall.

Ore pillar movement was measured by extensometer 22-246 installed in an uphole from the undercut elevation into the 22-134 stope to measure ground movement in the ore pillar adjacent to the 22-139 stope. Components of movement parallel and perpendicular to the ore pillar were obtained. Measurements at the center, mid height of the pillar showed that movement away from the footwall and towards the open stope occurred as mining progressed. Movements in the footwall area at the undercut elevation were larger.

Fill pillar movement was measured by extensometer 22-245 installed in an up hole from the undercut elevation with the first anchor in the footwall and the second one at the center, mid height of the fill pillar. Movements parallel to the fill pillar were measured as mining progressed.

fig 7.28

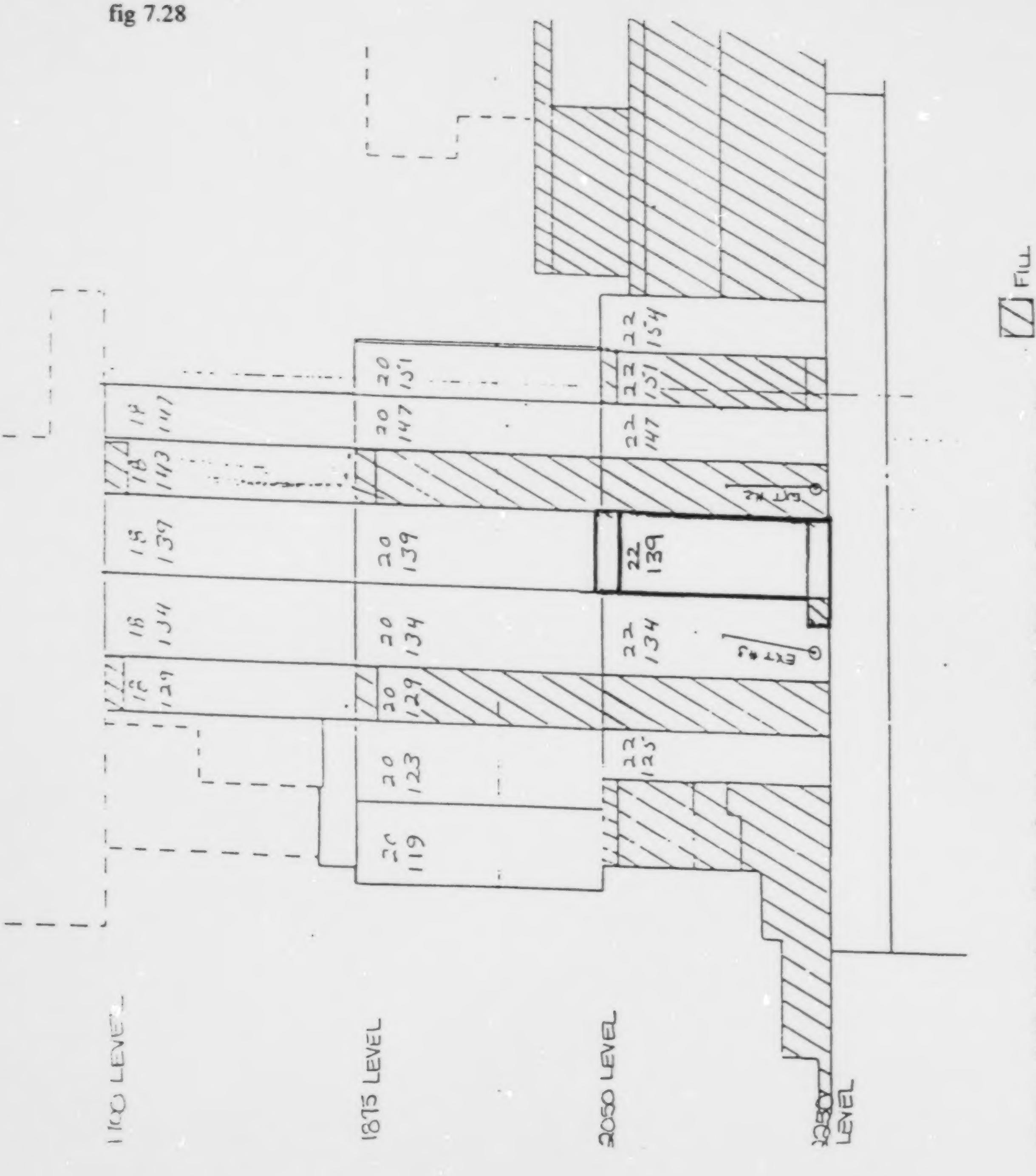

1700 LEVEL
1875 LEVEL
2050 LEVEL
2250 LEVEL
18 141
18 143
18 139
18 134
18 129
20 151
20 147
20 139
20 134
20 129
20 123
20 119
22 154
22 151
22 147
22 139
22 134
22 125
EXT #2
EXT #3
FILL

Figure 7.28 Section view of test stope location at Lockerby mine.

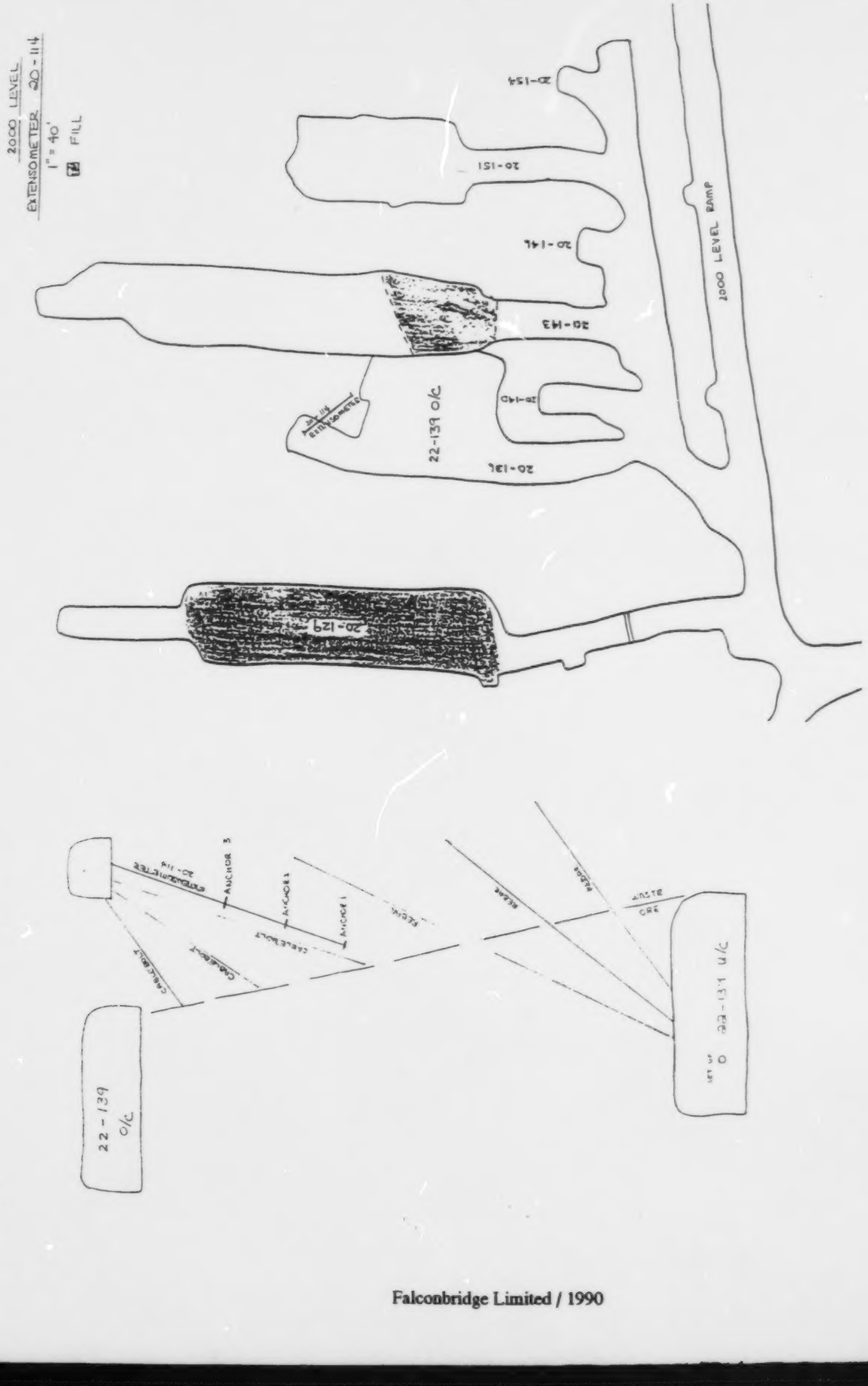

2000 LEVEL
EXTENSOMETER 20-114
1" = 40'
FILL
20-154
20-151
20-146
20-143
20-140
20-136
22-139 O/C
EXTENSOMETER 20-114
20-129
2000 LEVEL RAMP
20-114 EXTENSOMETER
ANCHOR 3
ANCHOR 2
ANCHOR 1
CABLEBOLT
CABLEBOLT
CABLEBOLT
REBAR
REBAR
REBAR
WASTE
ORE
22-131 U/C
SET UP O
22 - 139 O/C

Figure 7.29 Location of extensometer 20-114.

fig 7.30

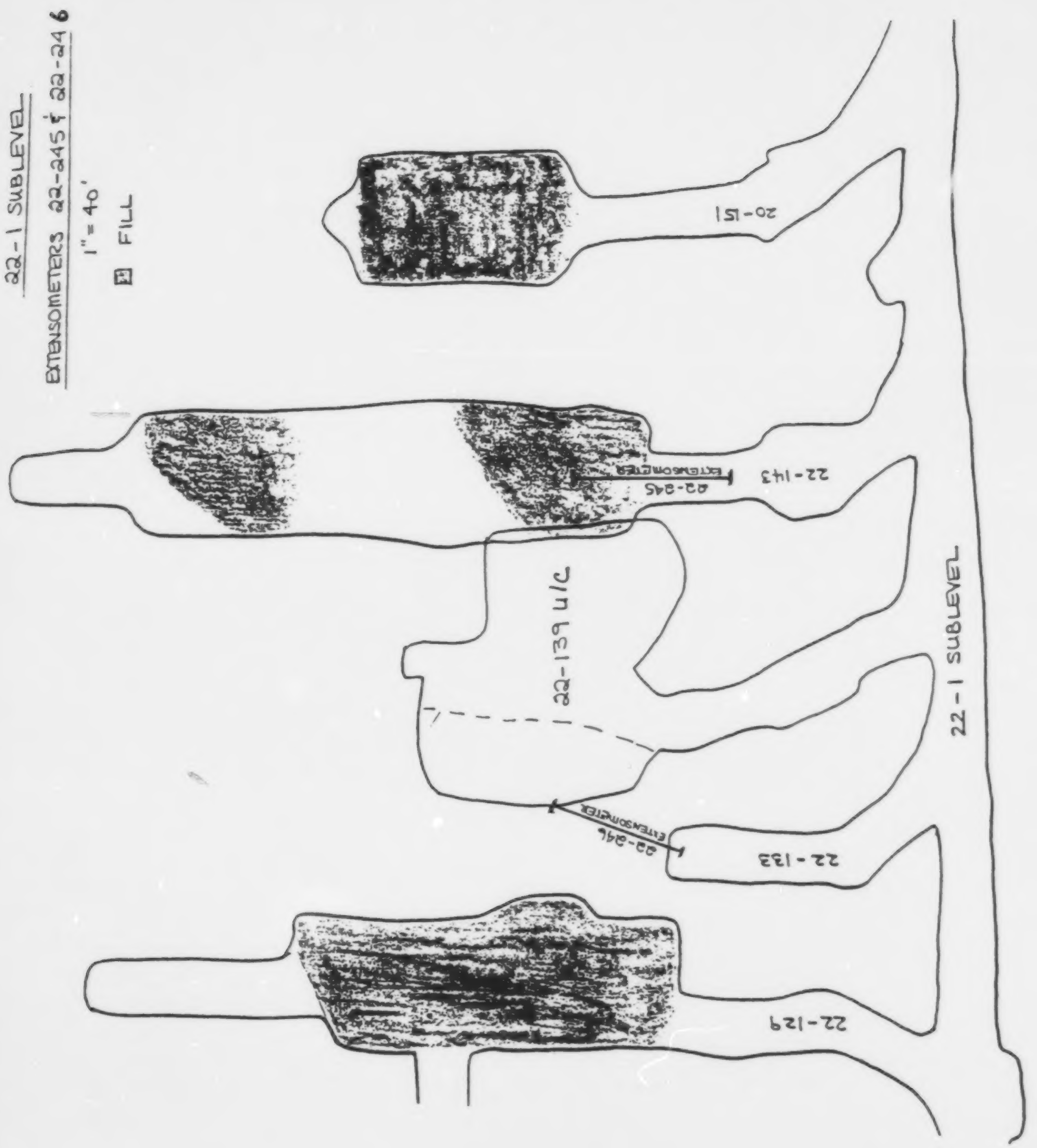

22-1 SUBLEVEL
EXTENSOMETERS 22-245 & 22-246
1" = 40'
▨ FILL
20-151
22-143
22-245
EXTENSOMETER
22-139 U/C
22-246
EXTENSOMETER
22-133
22-129
22-1 SUBLEVEL

Figure 7.30 Location of extensometers 22-245 and 22-246.

Visual Inspection: Visual inspection of the fill pillar during mining of the 22-139 stope indicated that a minimal amount of dilution originated from the fill pillar as a result of sloughing in small, isolated zones of fill. No signs of shear, tensile or deformation failure were observed.

Fill exposures in the east wall of the overcut and undercut of the 22-139 stope were profiled and cement contents were determined for each of the zones delineated. Cement contents were also determined at sampled locations within the fill pillar. The information gathered showed that the fill pillar is composed of a complex arrangement of zones which are segregated with respect to both cement content and particle size. In this case, a designed uniform bulk pour of 12:1 produced a fill pillar containing zones with cement-tailings fill ratios which varied between 44:1 and 1.3:1. Figures 7.31 and 7.32 show the segregation mapped in the over cut and the under cut.

Core Recovery: Samples of fill obtained when drilling the holes for dilatometer testing was used for laboratory testing of uniaxial compressive strength. Difficulties with core recovery in weakly cemented fill meant that only samples with a cement content of 5.3% or better could be recovered.

The results of the laboratory testing showed;

1. Uniaxial strength of samples tested were not entirely controlled by their cement contents. Between cement contents of 5% and 25% there is a general but poorly defined trend of increasing strength with increasing cement content. Samples in which the cement content was higher than 35% did not develop strength in proportion to their cement contents.

2. No unique relationship between stiffness and cement content could be determined. The range of stiffness values measured varied from 0.32 to 1.91 GPa, which is within the general ranges of values measured in situ by dilatometer testing.

It was concluded that cement content was only one of the parameters controlling fill properties. The segregation occurring during bulk pours results in variability in both cement content and particle size distributions within the fill. Segregation may sometimes be the controlling determinate of fill strength.

fig 7.31

FILL PROFILE : 22-139 STOPE OVERCUT

SAMPLE	TAILINGS/CEMENT RATIO	% CEMENT	NOTE
①	13.6:1	6.8	MATRIX COMPOSITION
②	15:1	6.3	
③	17:1	5.6	
④	5.44:1	15.5	
⑤	11.9:1	7.3	
⑥	2.12:1	32.1	

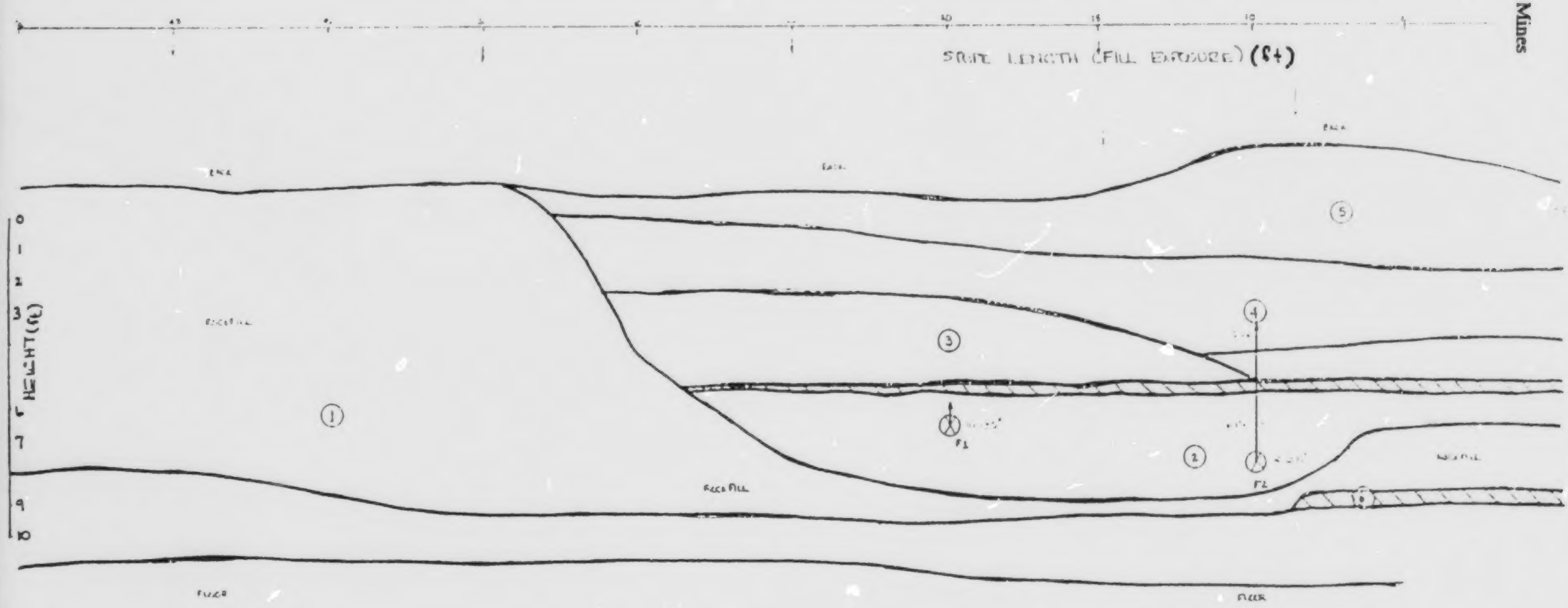

STOPE LENGTH (FILL EXPOSURE) (ft)
HEIGHT (ft)
0
1
2
3
6
7
9
10
①
②
③
④
⑤
⑥
ROCK FILL
F1
F2
FLOOR

Figure 7.31 Fill profile at over cut of 22-139 stope, showing fill segregation.

fig 7.32

FILL PROFILE : 22-139 STOPE UNDERCUT

SAMPLE	TAILINGS/CEMENT RATIO	% CEMENT
①	7:1	12.5
②	14.9:1	6.3
③	20:1	4.8
④	34.5:1	2.8
⑤	43.5:1	2.2
⑥	8.5:1	10.8

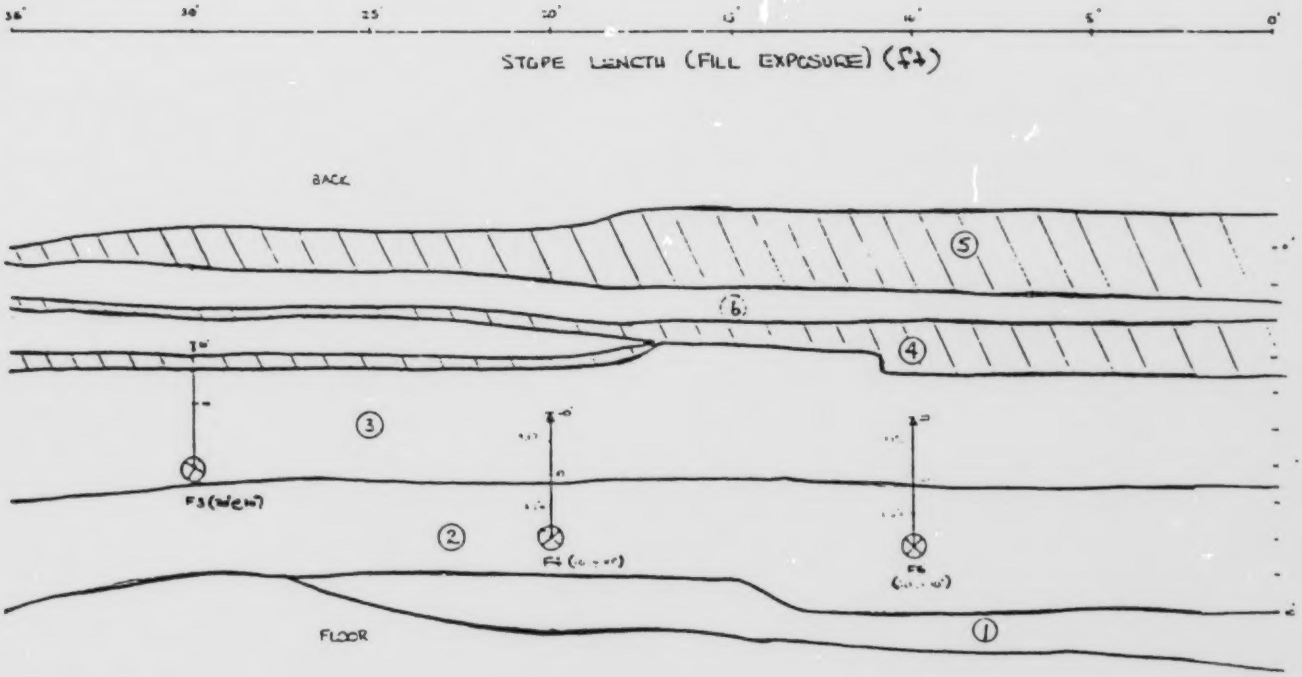

35'
30'
25'
20'
15'
10'
5'
0'
STOPE LENGTH (FILL EXPOSURE) (ft)
BACK
⑤
⑥
④
③
②
①
F3
F4
F6
FLOOR

Figure 7.32 Fill profile at under cut of 22-139 stope, showing fill segregation.

Blast Vibration Monitoring: A number of production blasts were monitored in conjunction with this program. Stopes 22-143 and 34-151 were monitored with sensors installed in the stope under cut. The monitoring equipment used consisted of;

Sensors - acclerometers BBN Model 525-150
Recorder - AM Radio Shack model or FM/AM TEAC R61 Model
Oscillograph - For providing hard copy seismograph for analysis.

The sensors were mounted on an aluminium block which was fixed to the rock surface with fast setting rockbolt resin. The sensors were mounted in the conventional way so that they were arranged in three mutually perpendicular directions and thus are capable of measuring the three components of planar wave motion. Monitoring of vibrations in cemented backfill were carried out behind the fill bulkhead for the 22-143 stope. The design tailing binder ratio for this stope was 12:1. The sensors were installed by cutting a horizontal hole 150mm by 200mm by 610mm long into the fill behind the bulkhead; a aluminum sensor mounting block with three accelerometers attached was set in a notch in the bottom of hole and backfill cuttings were packed around the instrument to stabilize it.

A manual analysis of the vibration data was made from the paper strip seismic traces. Monitoring was done at a scaled distance of about 8, which for 68 kg per delay is about 30m, from the blast, (scaled distance is calculated using the relationship $D/\sqrt{W}$; D is distance from blast, W is charge weight of explosive).

Table 7.10 details the results of the monitoring. The results, although inconclusive, indicate;

1. The backfill material approximates an elastic material more closely than would have been thought. This may be explained by the very short duration of the dynamic stresses and the high binder content of the fill. The frequencies of vibration measured ranged from approximately 25 to 75 Hz. This higher than expected and may be due to the high binder content.

2. No conclusive co-relationships were made between particle velocity and ground behavior during this program. It should be noted that the variations in static loading would be expected to vary widely with depth of fill and this has a major impact on the vibration levels (dynamic tensile loads) that can be sustained prior to failure.

Stope Monitored	Location of Sensors	Scaled Distance	Peak Particle Vel. in/sec	Frequency Hz
22-139	22-143	7	6.9	25-30
	behind bulkhead	7.5	4.4	-
		9	3.8	50-75
22-139	22-143	6.5	3.8	50-75
	behind bulkhead	18	2.9	-

Table 7.10 Summary of Blast Vibration Monitoring of Hydraulic Fill at Lockerby Mine.

Chapter 8
Binder Alternatives

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    8.3.1 Portland cement hydration
    8.3.2 Ferrous slags
    8.3.3 Non ferrous slags
    8.3.4 Fly ash

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        - 8.6.1.3 Effect of requenching and lime enrichment
        - 8.6.1.4 Varying the proportion of slag replacement
        - 8.6.1.5 Effect of acidulation on slag performance
        - 8.6.1.6 Addition of gypsum, lime and anhydrite
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- 8.7 Anhydrite as a Hydraulic Fill Binder Alternative
    - 8.7.1 Test Work
    - 8.7.2 Results

- 8.8 Falconbridge Sudbury Operations Experience with Ferrous Slag as a Hydraulic Fill Binder
    - 8.8.1 Problems with the fineness of slag
    - 8.8.2 Problems with the curing rate of slag
    - 8.8.3 The solutions

8.1 INTRODUCTION

This chapter presents the results of a considerable volume of test work on binder alternatives conducted under the auspices of the project. These binder alternatives were cementitious and pozzolanic materials that may be used to completely or partially replace normal portland cement as a backfill binder. The incentive for this work was to identify binders which would permit the placement of lower cost and or higher strength backfills. In mines employing consolidated fill, up to 50 percent of the overall operating mining costs can be due to the cost of portland cement, depending on availability and transport distance (Dhar et al 1983, Thomas 1973). Alternative binders have become common in the concrete industry and in mining operations around the world. This project was initiated to determine how the binder alternatives available in Ontario would perform with backfill materials typical of the Ontario mining industry.

Binder alternatives for hydraulic fill and rock fill were considered separately. Falconbridge, Sudbury operations were responsible for test work on hydraulic fill and Kidd Creek Mines were responsible for test work with rock fill.

The following binders were tested;

1. Normal portland cement (base case binder against which others are judged)
2. Ground ferrous blast furnace slags
3. Ground copper slag
4. Type C fly ash
5. Type F fly ash
6. Anhydrite

The slag and fly ash binder alternatives are known as "pozzolanic" materials. Technically, a pozzolan is defined as "a siliceous or alumino-siliceous material that in itself possesses little or no cementitious value but that in finely divided form and in the presence of moisture will chemically react with alkali and alkaline earth hydroxides at ordinary temperatures to form or assist in forming compounds possessing cementitious properties" (ASTM C219, as quoted in Nieminen & Seppänen 1983). Another source (Thomas 1973), defines a pozzolan as "a material containing reactive silica which forms stable, cementitious, complex hydrated calcium silicates in the presence of lime and water". Typical examples of pozzolans are PFA (pulverised fuel ash) or fly ashes from coal or peat, ground slags such as copper reverberatory furnace slag, lead blast furnace slag, quenched copper converter slag, iron ore blast furnace slag, nickel slag, anhydrites, silica fume, monolithic packing materials, pyrrhotite, natural volcanic ash, pumicite, opaline shales and cherts, burnt clay and various chemicals with patent-protected formulae.

The disposal of slags and fly ash is an environmental problem for their producers. Therefore, there is a "convergence of interests" between slag or fly-ash producers on the one hand and mining companies on the other where these materials can be profitably employed as low-cost substitutes for portland cement in cemented backfill preparations (Manca et al 1983).

There has been extensive research for many years carried out around the world on various binder alternatives for backfill prior to this project. This previous work, and experience from the local concrete industry was used as the basis for establishing the project test program. The specific aim of the project with respect to binder alternatives was to identify what potential low cost binders were available locally in Ontario and to determine how they would perform with backfill materials and systems typical in Ontario mines.

The work conducted for the project has confirmed that slag and flyash binder alternatives available in Ontario behave as previous work by CANMET, (reference to various CANMET reports principally Douglas & Malhotra), suggested and that they show potential for reducing the cost of placing consolidated backfill in mines.

Anhydrite is used in the coal and soft rock mining industry as a component in monolithic packing materials. It has some potential as an binder alternative for use in hard rock mine backfill. As part of a general subcontract to the project, which investigated the potential of monolithic packing materials, test work was conducted to determine just how anhydrite would perform as hydraulic fill binder. The work was only preliminary and used anhydrite in proportions which would be uneconomic in practice. It did show that anhydrite could produce strong hydraulic fill.

8.2 IMPORTANT PROPERTIES OF A BINDER

Assessing the suitability of a binder principally involves evaluation of;

1. Ultimate strength developed.

2. Rate of strength development.

3. Physical interaction with the backfill particles.

4. Cost.

In addition, the particle size, the storage, handling, mixing and effect on other mine systems are important considerations in the assessment of a binder.

The interaction between mine scheduling and the performance of a binder is a fundamental consideration in the selection of the type of backfill binder and the amount of binder used. In general most binder alternatives available will attain their strength at a slower rate than portland cement. The mining method and schedule will determine if this slower rate is important or not. The cost savings possible with some binder alternatives may be significant enough to justify the modification of the method or the schedule. The point being that binder alternatives can not be considered as just a lower cost cement, they should be considered as an integral part of the mining system. A binder's suitability is material, mining method, and mining schedule specific.

The strength of a binder is generally measured in terms of unconfined compressive strength. In order to characterize the rate of strength development unconfined compression testing was carried out after various curing times. Typically testing was done at 4, 7, 14, 28, and 56 days of curing, which is consistent with previous work on backfill and standard concrete practices.

The particle size of a binder can be of great importance. In general, as the fineness of grind of a pozzolanic or cementitious material increases so does the reactivity of the product and hence the rate of strength development. This report discusses the particle size of binders in terms of Blaine size. Blaine size is a measure of the surface area available per unit weight (cm^2/g). It is a convenient way of characterizing the size distribution of very fine materials with a single number. Typical Blaine size for normal portland cement is around 3600, while some of the slags tested had Blaine sizes around 4600 cm^2/g.

The cost of a binder is of overriding importance in considering its suitability. The high cost of portland cement to the Northern Ontario mining industry has been the greatest incentive for research into binder alternatives which may be cheaper to produce or may be found locally resulting in reduced transport costs. Portland cement costs in Northern Ontario are in the range $100 to $150 per tonne (1990 dollars), depending upon locality and size of the supply contract. A large blast hole mining operation producing 1M tpa and requiring considerable tonnages of consolidated fill would consume in the region of 20 000 tonne of binder per year at a cost of $2M to $3M per year. The incentive for use of a cheaper alternative binder is obvious.Binder costs may be compared in terms of dollars per effective kPa of strength supplied.

8.3 CHEMISTRY

8.3.1 Portland Cement Hydration an Overview of the Theory

Portland cement consists essentially of crystalline compounds of calcium combined with silica, alumina, iron oxide, and sulphate (a mutli component system of CaO - MgO - SiO_2 - Al_2O_3 - Fe_2O_3 - SO_3). These minerals are unstable in water and if wetted will begin to dissolve or ionize, i.e hydrate. The ionized species will form hydrated products with low solubility which precipitate out of solution forming a cementitious gel. The chemical reactions of hydration of portland cement compounds are complex and often do not proceed to completion. The main product of the reactions of the silicate minerals is calcium silicate hydrate (CSH) which accounts for about 60% of the total solids in a fully hydrated portland cement and is responsible for most of the strength development. The other main product of the hydration of calcium oxide minerals is calcium hydroxide which contributes little to the strength.

The chemical components of portland cement are present in the crystalline form as various calcium silicate, aluminate and ferrite minerals. The principal minerals are;

1. Tricalcium silicate $3CaO.SiO_2$ - C_3S

2. Dicalcium silicate $2CaO.SiO_2$ - C_2S

3. Tricalcium aluminate $3CaO.Al_2O_3$ - C_3A

4. Tetracalcium aluminoferrite $4Ca.Al_2O_3.Fe_2O_3$ - C_4AF

These minerals are formed during the manufacture of the portland cement clinker when CaO combines with the acidic components of the raw mix during the burning operation in the kiln. The portland cement minerals will all hydrate easily, releasing the CaO, SiO_2, Al_2O_3, and Fe_2O_3 required to form the cementing calcium silicates, aluminates and ferrites.

A gross simplifcation of the complex portland cement hydration reaction is given below, (cement chemistry notations are used. C = CaO, S = SiO_2, A = Al_2O_3, F = Fe_2O_3, H = OH)

$$C_3S + aq = CSH + CH$$

The same general reaction occurs with other silicate minerals, and with aluminate and ferrite minerals to produce CA or CF hydrates. The portland cement reaction is a lime, CH, producing one. This is an important point when considering the chemistry of pozzolans.

Hydration reaction products of the aluminate, iron oxide and sulfate minerals also contribute strength to the cement, especially in the early stages of hydration. The trisulfate calcium alumina hydrate known as ettringite is usually the first product to precipitate out, and is mostly responsible for the initial set and very early strength of portland cement. This alumina sulfate interaction may have an iron sulfate analog interactions which also results in strength development. Small amounts of calcium sulfate, usually in the form of gypsum are added to portland cement clinker during the grinding operation to control these initial reactions and the early set.

A ternary diagram of the system is shown in figure 8.1. Also indicated is where the pozzolanic materials which may be used as alternatives to portland cement are located in the system.

The hydration of portland cement minerals produces excess quantities of calcium hydroxide, $Ca(OH)_2$, which is the product required to make pozzolanic materials react. The silicate minerals in a pozzolan will react with excess calcium hydroxide to form new calcium silicate minerals adding to the strength of the bonds formed by the hydration of the original portland cement. A true pozzolan is required only to provide additional silicate minerals in a reactive form which will allow the formation of additional calcium silicate products.

Table 8.1 below shows the typical chemical composition of the binder alternatives tested during the project.

	Portland Cement	Ferrous Slag "A"	Ferrous Slag "B"	Copper Slag	Type C Fly Ash	Type F Fly Ash
CaO	65	33	35	5	17	7
SiO_2	20	38	40	33	47	47
Al_2O_3	4	8	7	3	19	25
Fe_2O_3	3	trace	1	53	7	9
MgO	3	16	12	-	3	3
SO_3	3	1	4	-	7	1
Other	2	4	1	6	-	8

Table 8.1, Composition of portland cement and various pozzolanic materials, %.

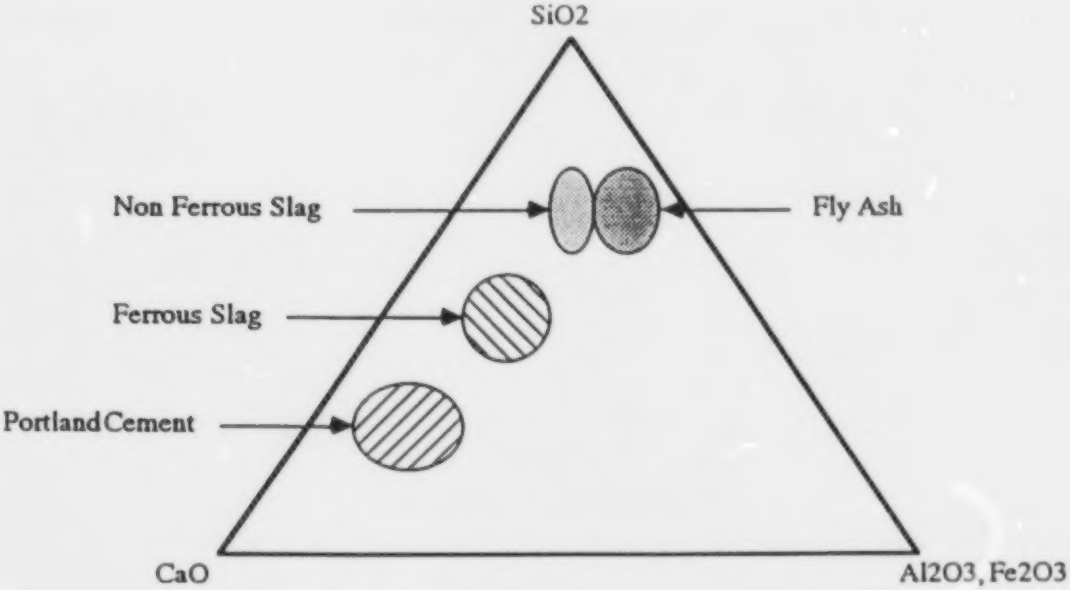

SiO2
Non Ferrous Slag
Fly Ash
Ferrous Slag
Portland Cement
CaO
Al2O3, Fe2O3

Figure 8.1, Ternary diagram of cement and pozzolans

8.3.2 Ferrous Slags

Ferrous slag is a by product from iron or steel blast furnaces. The molten slag is rapidly quenched from high temperature, either in water or air and water, so that most of the lime, magnesia, silica and alumina can be held in a glassy non crystalline state. When ground to a fine size (4000 to 6000 cm^2/g) this slag is capable of developing satisfactory cementitious and pozzolanic properties. A slag quenched in water is called granulated due to the sand sized particles produced, and slag quenched in air and water is called pelletized due to the pellet sized particles produced. A granulated slag will generally contain a higher glassy content. A slag which is allowed to air cool slowly will be composed of materials in their crystalline form and will, if finely ground, exhibit only weak pozzolanic activity. The reactivity of a slag depends upon the minerals being present in a glassy unstable state. In their crystalline state the same minerals will not be reactive or only slightly so. The ferrous slags tested in this project were granulated slags.

The properties of slag that influence its reactivity are;

1. The degree of vitrification (glass content). Minerals present need to be in a non crystalline reactive state.

2. The chemical composition.

3. Mineralogical state. Different mineral forms of the same chemicals are less stable and more reactive than others.

4. Fineness of grind. Generally the finer the grind the higher the reactivity.

A ferrous slag is normally considered to be a pozzolan, requiring the hydration products of portland cement, e.g $Ca(OH)_2$, for it to show any strength development. The slags tested in this project were not true pozzolans. They showed some inherent cementitious ability. This is attributed to the high CaO content of the slags and their glassy, reactive state.

The normal reaction of a pozzolanic material is given as;

$$CH + S + aq = CSH$$

This is a lime, CH, consuming reaction. The lime in a true pozzolanic reaction would come from the hydration of portland cement. All a slag pozzolan has to provide is a sufficiently reactive supply of S. Although this CSH producing reaction is the principal reaction, cementitious products are also formed as a result of the chemical reactions between lime and alumina or iron oxide. The sum of the SiO_2, Fe_2O_3, Al_2O_3 content of a slag is an indicator of the total potential for formation of cement products. Typically these products consist of calcium aluminate hydrates or calcium aluminoferrite hydrates. If sulphate is present then calcium sulphoaluminate and sulphoaluminate ferrite hydrates may also form. Thus the composition of the end products of portland cement reactions and pozzolanic reactions are essentially the same. The only difference is in the rate at which these reactions occur.

The importance of mineralogy and degree of non crystalline state is more important to the reactivity of slag than its chemical composition. Mineralogy and degree of crystallinity is a function of the thermal history and the process by which it is produced. Two slags with the same chemical composition will show different reactivity because different thermal history. The details of the chemistry are complex and not well understood. In practice it means that each slag is a unique product and will behave in a unique manner with a particular fill.

Slag cements are of interest for mine backfill because they are available at a lower price than portland cement and because they have the potential to provide a higher strength product. A slag may contain a higher proportion of SiO_2, Al_2O_3 and Fe_2O_3 than portland and so we expect that a greater quantity of CSH CAH and CFH will be formed and less low strength $Ca(OH)_2$ crystals.

The ferrous slags tested in this project were found to be inherently cementitious and not true pozzolans. They posses sufficient CaO content to supply their own lime. The rate of hydration of the CaO from the glassy state in these slags is however very slow compared to portland cement. Activation of the slag reactions by the addition of small amounts of free lime was tried. This increased the alkalinity of the system and provided some additional Ca. It is well established in the concrete industry that pozzolanic reactions can be accelerated by temperature, and by the presence of alkalis and sulphates. These activators provide some free Ca for initial reaction with the slag silicates for early hydration of calcium silicates and also quickly raise the pH of the system to improve the rate of hydration.

8.3.3 Non Ferrous Slag

The non ferrous slag tested in this project was Kidd Creek copper slag. The same principles apply to copper slag chemistry as to ferrous slag, although the details are even less well understood. All the important factors which are known to affect ferrous slag were tested on the copper slag, e.g. effect of quenching, fineness of grind, amount and type of activator. Different types of activator were tested on the copper slag; lime, anhydrite, gypsum, and portland cement.

The difference in the chemistry of copper slag is that it may be considered a true pozzolan because of its low CaO content. It provides additional strength to portland cement by reacting with the excess calcium hydroxide product of portland hydration reactions to produce additional calcium silicate hydrates. This additional strength development implies that the slag can replace some of the portland cement which would normally be required. The replacement can not be too great else there may not be sufficient portland reactions products available to make full use of the available silicate minerals in the slag. These silicates in the slag should be in a state where they are sufficiently unstable to be reactive. This implies some form of glassy state and so the effect of quenching the slag as is done for ferrous slags was tested. It is assumed the fineness of grind might play some role in the reactivity of the slag, and this was tested also.

8.3.4 Fly Ash

Fly ash is produced from burning pulverized coal in power stations. It is removed by mechanical collectors or electrostatic precipitators as a fine particulate residue from the combustion gases before they are discharged into the atmosphere. The disposal of this material has been in tailings dam structures, and the significant quantities produced have raised environmental concerns.

Two types of fly ash are recognized as of interest as binder alternatives. Type C is a high lime content ash. It often shows some cementitious properties, i.e. is able to hydrate and develop strength without an activator and is thus not a true pozzolan. Type F is a lime poor ash which is truly pozzolanic and requires an activator to start hydrating. Type C ash is produced from burning lignite or sub bituminous coal. Type F ash is produced from burning anthracite or bituminous coal. Fly ash is a glassy spherical material.

The chemistry of hydration of fly ashes is again similar to that of the slags. Calcium hydroxide produced by the hydration of portland cement will react with the Si, Al, and Fe oxides of the ash to form new cementitious products. Type C ashes are capable of supplying their own Ca and thus are cementitious in their own right. As with slag the mineralogy of the material is just as important. The constituent minerals must be in their non crystalline, glassy state for the ash to have hydraulic potential. High Ca content ashes may have a portion of their CaO present as C_3S (same mineral as in portland cement), free lime and other minerals which are present in their crystalline state and capable of hydration. These ashes may be very reactive.

Typically fly ash is used only as a partial replacement for portland cement, up to 60%, the remaining portland being required to supply the CaO for formation of the calcium silicates, aluminates and ferrites.

Fly ash is produced essentially for free (its an unwanted waste product anyway) it is competitive in price against the slag products which require grinding, even though significant proportions of portland cement are still required to make it work.

Both type C and type F ash were tested for this project. Type C ash, being inherently cementitious results in a much faster strength development than the type F ashes. Type C ashes are therefore of most interest to the mining industry.

8.4 PREVIOUS WORK

The test work on binder alternatives under the auspices of this project was based upon a considerable volume of previous work conducted in Canada and around the world on a great variety of alternatives to portland cement. The concrete industry has been aware of and using slag and fly ash materials for 50 years, so much of the theory of their use is well understood. In backfilling operations the aggregate size and distribution, method of mixing and placement, and for hydraulic fill the pulp density are all very different to that found in the well engineered products of the concrete industry. So the parameter for the economic use of these alternatives must be re evaluated specifically for backfilling operations. This is not an easy task because backfilling operations are inherently material and site specific; no two mining operations have the same backfill materials with which to work, the same distribution restrictions or more importantly the same mining schedule parameters. The project aimed to test some of the binder alternatives available to the Ontario mining industry with backfill materials and systems typical of the local industry. It was intended to identify the more promising of the alternatives and provide some guide to parameters for their use. Individual operations can use these parameters as a starting point for their site specific testing.

8.4.1 Non Ferrous Slag Investigations

Non ferrous slags are well established as pozzolanic replacements of portland cement in mine backfill. Probably the best known application is at Mt Isa, Australia. Here quenched copper slag is used as a partial replacement of portland cement, (Thomas, Cowling, 1978). The O'okiep Copper company in South Africa is another well known user of non ferrous slag. Here a cementitious modified copper slag is used with hydraulic tailings fill. Activation of this slag requires lime but no portland cement (Atkinson et. al., 1989).

In general non ferrous slags are true pozzolans (O'okiep modified copper slag being a major exception) with low CaO content. Table 8.2 shows the composition of various non ferrous slags including the Kidd Creek copper slag tested in this project.

	Portland Cement	Mt Isa copper	Falco copper/nickel	Kidd Creek Copper	O'okiep mod. copper
CaO	65	7.9	2.2	5	14.3
SiO_2	20	35.5	35.5	33	40.1
Al_2O_3	4	3.6	4.3	3	5.9
Fe_2O_3	3	36.0	40.0	53	27.1
MgO	3	1.9	3.8	-	7.6
SO_3	3	0.9	-	-	0.4

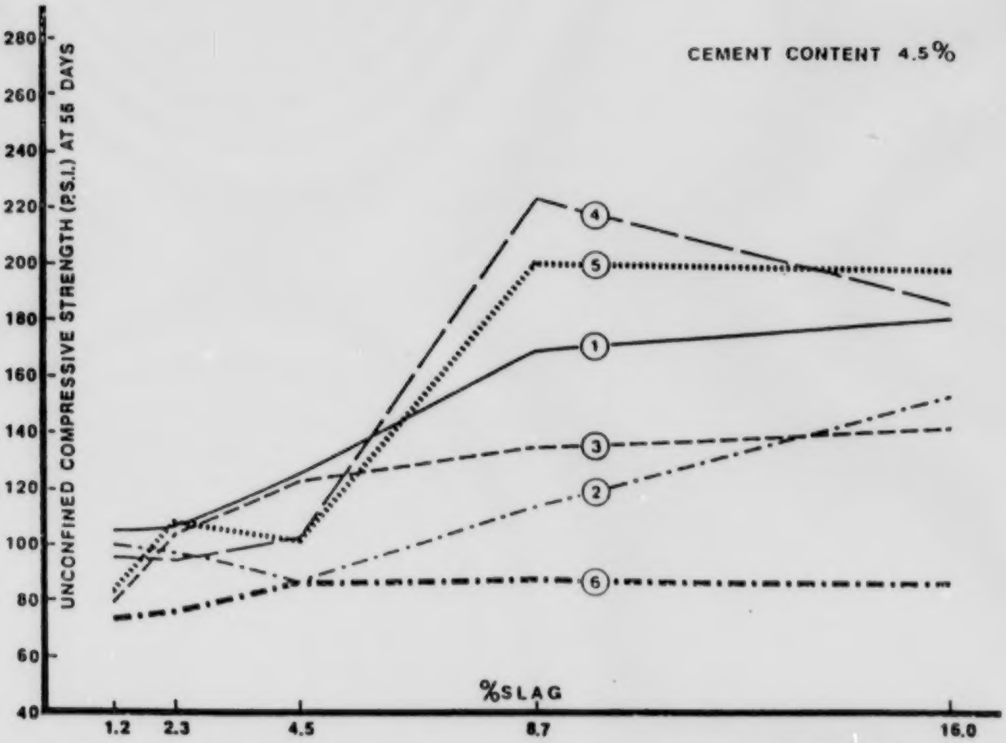
Table 8.2, Composition of various non ferrous slags, % by weight

In Ontario considerable work has been done by Douglas and Malhotra of CANMET investigating the potential of locally produced copper and nickel slags (various papers by Douglas and Malhotra available through CANMET describe extensive research). Falconbridge (McGuire, 1978) and INCO (Barsotti, 1978) have also conducted their own test work. The generally conclusion from all this test work is that non ferrous slag available Ontario did exhibit pozzolanic activity and can be used to partially replace portland cement in mine backfill. Their use is a matter of economics in competition with other binder alternatives.

Successful efforts in using nickel slags were conducted by INCO Ltd. as reported by Barsotti (1978).

Here, air-cooled nickel reverberatory slag ground to 3500 to 4000 Blaine had good pozzolanic properties in spite of the fact that the glass content was low (i.e. below 20 percent). Vitrified nickel reverberatory slag ground to the same fineness produced higher strength, but the cost of granulating the slag was too high. The addition of calcium and magnesium carbonates to the molten nickel slag prior to vitrification improved cementing quality but was uneconomical. The mix seemed optimal when the amount of pozzolanic slag was twice the amount of cement. For example, a mix of 4 percent cement and 8 percent slag was very successful. Tests at INCO indicated that these percentages plus 88 percent tailings would produce a fill strong enough to stand unsupported up to 61 m and would successfully resist blast vibrations. Further tests reported by Barsotti in the same article showed the effects of varying quantities of nickel slags and ground quartz on the 56-day unconfined compressive strength of backfill. The results can be seen in Figure 8.2. Tests were conducted on tailings / slag samples poured at 75 percent pulp density by weight and cured at 80 degrees Fahrenheit and at 100 percent humidity. The graph in Figure 8.2 shows that the highest strength achieved was over 220 psi at a slag content of 8.7 percent. Further increases in slag resulted in lower strength.

Falconbridge conducted test work involving finely ground granulated nickel - copper electric furnace slag. This was tested for use as a partial replacement of portland cement. Results indicated that a satisfactory product could be produced, but the cost of producing it was uneconomic. In this test work it was found that granulation was required to develop the pozzolanic activity, fine grinding alone was not sufficient.



CHEMICAL COMPOSITION OF THE MATERIALS TESTED

MATERIAL	Fe	S	SiO_2	CaO	MgO	Al_2O_3	$\frac{\%Al_2O_3 + CaO + \%MgO}{\%SiO_2}$ HYDRAULIC MODULUS
(1) C.C. DUMP SLAG AIR COOLED	36.0	1.5	35.5	2.25	2.50	5.05	0.2761
(2) C.C. DUMP SLAG WATER QUENCHED	35.8	1.4	36.8	2.30	2.45	5.55	0.2799
(3) CONISTON DUMP SLAG, AIR COOLED	35.8	1.5	30.8	7.00	2.45	6.00	0.5016
(4) CONISTON DUMP SLAG WATER QUENCHED	34.4	1.4	31.0	7.05	2.50	6.50	0.5177
(5) C.C. DUMP SLAG + 14% LIMESTONE WATER QUENCHED	32.9	1.3	32.8	9.40	2.45	5.85	0.5396
(6) GROUND QUARTZ	0.58	--	87.7	0.27	0.029	1.10	0.0160

Figure 8.2, UCS strength of hydraulic fill achieved with various binders, work by Barsotti, 1978.

8.4.2 Ferrous Slag and Fly Ash Investigations

There has been extensive research done for the concrete industry on the use of ferrous slag and fly ashes as pozzolanic materials. Both materials are well accepted as a binder alternatives, with the chemistry and mechanics of their activation being reasonably well understood. The use of these pozzolans in the mining industry is simply a transfer of technology.

CANMET researchers have been active in investigating ferrous slag potential in hydraulic backfill. Douglas and Malhotra (1987) found that lime activated granulated ferrous slag could replace portland cement in backfill and perform as well or better than the portland cement. Underground field tests were also conducted. The slag produced a satisfactory fill.

8.4.3 Activators

Various binders have different rates at which they will develop strength, this is controlled by the chemical reactivity of the system. Pozzolanic materials will characteristically show a slower strength development than portland cement and will commonly require the addition of an activator to make them viable alternatives. The slow reactivity of pozzolanic type materials is a result of the slow hydration of ions from the glassy state compared to the hydration from the crystalline state in portland cement.

An activator will commonly be an alkali or sulfate. The alkalinity is required to aggressively etch and dissolve the pozzolan glass. Vienneau (1983) conducted tests with seven potential activators for ferrous slag. The substances tested were; sodium carbonate (soda ash), calcium oxide (quick lime), calcium hydroxide (hydrated lime), sodium chloride (salt), calcium chloride (rock salt), sodium sulphite and sodium hydroxide. There are other activators available for ferrous slags, such as portland cement itself or gypsum, but these were not tested by Vienneau. The amount of activator added was equivalent to 5% by weight of the amount of slag added. Results indicated that sodium hydroxide, quick lime and hydrated lime were the most effective activators. Because of the high cost of sodium hydroxide ($840/ ton in 1983 Canadian dollars) as compared to quick lime ($66/ ton), lime was found to be the most attractive activator. In fact, lime is still today the most popular activator used for backfill mixes containing slag as a binder.

Some other activators which consist of commercially produced chemical mixtures are also known under various trade names. One example is "Conbex Backfill" (Foseco Canada / Technik Mining). This is an admixture that is custom blended to suit each site and each specific application within one site. This allows the activator to be ideally formulated to react with the varying chemistry from one backfill application to another.

A reasonably reactive slag or ash will create its own alkalinity as hydration proceeds, but this may be slow, so we still tend to add an alkali activator at the beginning. In the concrete industry were pozzolans are commonly used only for partial replacement of portland cement, the portland itself acts as a suitable activator.

8.4.4 Effect of Grind

The grind of a binder can markedly affect its cementing ability. For example, at Mt. Isa Mines in Australia, experiments showed that grinding to a minimum of 90 percent minus 400 mesh maximizes the cementing ability of copper reverberatory furnace slag (Ashby & Hunter 1982). The influence of varying the grinding fineness on the strength level achieved by a slag binder is also reported by the Standard Slag Cement Co. (1986). When using slag, the coarsest mix possible without compromising the reactivity or the segregation potential of the binder is the most economical because of lower grinding time and higher production rate. On the other hand, finer grinds tend to result in higher ultimate strengths due to the greater surface area available for binder reaction. This could reduce the overall quantity of binder needed to produce a given strength and could offset the costs of additional grinding. The effect of the various factors on curing time and strength development can only be determined by laboratory and field tests that incorporate site-specific conditions.

Slags because of their lower hydraulic reactivity will require a finer grind than a typical portland cement. Typically a slag has a fineness around 4000 to 5500 cm^2/g (Blaine size), a portland cement will have a fineness around 35000 cm^2/g.

The fly ashes tested in this project did not need grinding. They are used as is from the coal power stations. This is why they are so cheap, the principle cost being for their transport and storage. It is possible to use a pulverised fly ash which has been subject to some coarse size reduction to improve reactivity but this involves additional cost and is avoided if the ash is sufficiently reactive.

8.4.5 Other Information

There are many publications that discuss supplementary or alternative cementing materials and admixtures for concrete and/or backfill. Most of these are directed toward replacing cement in concrete, although all of them are useful in the search for cement substitutes in backfill. It should be noted, though, that experts in the concrete industry who were contacted by Falconbridge personnel to comment on the potential performance of binder alternatives in backfill were not able to provide definitive answers. They felt that the marked difference in pulp density of poured fill as compared to concrete could affect performance of the binder alternatives. Therefore, any information relating to binder alternatives in concrete should be carefully evaluated before it can be applied to backfill.

The reader can refer to the following publications for background information on binder alternatives in concrete and / or backfill, if required: Berry & Malhotra 1987; Berry 1987; Diamond, 1980; Douglas 1987; Douglas et al 1986; Douglas & Malhotra 1985 & 1987; Garòfalo & Giovambattista 1985; Gertman et al 1978; Hannaford et al 1988; Hooton 1987; Hopkins & Beaudry 1989; Hwang & Lin 1986; Ionescu & Ispas 1986; Kasami et al 1983; Kheok 1981; Malhotra 1987a & 1987b; Manca et al 1983 & 1984; McGuire 1978; Mehta 1983, 1984, 1985, 1987a & 1987b; Metso et al 1983; Nantel & Lecuyer 1983; Nieminen & Seppänen 1983; Pandey 1984; Phillips 1981; Swain 1973; Talling 1985; Thomas 1973; Thomas & Cowling 1978; Vienneau 1983; and Yu & Counter 1986 & 1988.

8.5 HYDRAULIC FILL BINDER ALTERNATIVES, RESEARCH WORK BY FALCONBRIDGE LIMITED

As part of this project Falconbridge Limited, Sudbury operations were responsible for conducting test work to investigate binder alternatives suitable for use with hydraulic fill. The work was conducted on Falconbridge conventional classified hydraulic tailings fill from the Strathcona mill. A range of five binders were tested in initial work. The most promising binder alternatives, both ferrous slags, were selected for further investigations. The performance of these slags with both portland cement and lime as activating agents was tested. Finally an underground test pour of slag "A" which had proved to be the best alternative was conducted, and the economic advantages of its use considered.

8.5.1 Binders Tested

Five binder alternatives were tested.

1. Type C fly ash (Detroit)
2. Ferrous slag cement, "A", from Algoma Steel Limited in Sault Ste Marie, Ontario.
3. Ferrous slag cement, "A" activated with lime.
4. Ferrous slag cement, "B", from Stelco Limited in Hamilton, Ontario.
5. Ferrous slag cement, "B" activated with lime

These five binders were selected for their availability in Ontario and lower unit costs than portland cement. Their chemical composition is given below in table 8.3. Other alternatives such as Silica Fumes, Monolithic packing materials, and Type F fly ash were reviewed but for reasons such as; poor availability, handling difficulties, poor initial test results or costs, they were not evaluated in detail with hydraulic fill.

	Portland cement	Slag "A"	Slag "B"	Type C Fly Ash
CaO	65	33	35	17
SiO_2	20	38	40	47
Al_2O_3	4	8	7	19
Fe_2O_3	3	trace	1	7
MgO	3	16	12	3
SO_3	3	1	4	7
S	0	0	0	0
$Na_2O + K_2O$	1	1	trace	0
MnO	0	1	0	0
TiO_2	0	0	trace	0
SrO	0	0	trace	0
Mn_2O_3	0	0	1	0
Other	1	2	0	0
Blaine Size (cm^2/g)	3500	4600	4200	4000 - 7000

Table 8.3, Chemical composition of binders tested with hydraulic fill, %.

8.5.2 Laboratory Testing

Classified tailings from Falconbridge, Sudbury Operation's Strathcona mill were used as the basis for all hydraulic fill binder tests. The following laboratory conditions were kept consistent;

1. Fill slurry prepared at 68% solids by weight.
2. Curing humidity 100%.
3. Curing periods from 1 to 56 days.
4. A 24 hour open air drying time prior to testing.
5. Sample size 3 inch by 6 inch undrained cylinders.
6. Unconfined uniaxial compression testing.
7. Binder addition rates of expressed in tailing:binder by weight

8:1 = 11%
16:1 = 5.9%
32:1 = 3.0%

The fine particle size of mill tailings suggests that there is sufficient surface area available for some chemical interaction between the binder and the tailings minerals. In fact the remarkably good performance of ferrous slags in these tests suggested that there is some positive interaction with Strathcona mill tailings. The performance of a binder may vary with different tailings or sands used as a hydraulic fill. The mineralogy of Strathcona Mill tailings used in these tests is presented in table 8.4. Figure 8.3 shows the size distribution.

Mineral	% by weight
Quartz	24
Feldspar	33
Pyroxene	22
Amphibole	12
Phyilosilicates	4
Epidote (other silicates)	3
Magnetite, Chromite, Ilmenite	1
Sulfides (mostly pyrrhotite)	1

Table 8.4, Mineralogy of Strathcona classified mill tailings.

It is very important to consider time and curing rates in the comparative study of binders for use in backfill. Different binders develop strength at different rates, which may be varied with chemical activators. The strength development rate may have a significant impact on mine scheduling. Short term strength is important to those mines which need to fill a given void and then return quickly with men and equipment. This early strength is also important to mines that pour backfill plugs in the bottom of blasthole stopes. These plugs then retain the bulk of the stope fill. Early strength in this plug allows the mine to complete the remainder of the stope backfilling on a shorter schedule. Final strength is often the basis for design of fill composition. It is the final fill quality that will allow a mine to efficiently mine next to a previously backfilled stope. High fill quality will minimize fill dilution and can improve local ground conditions.

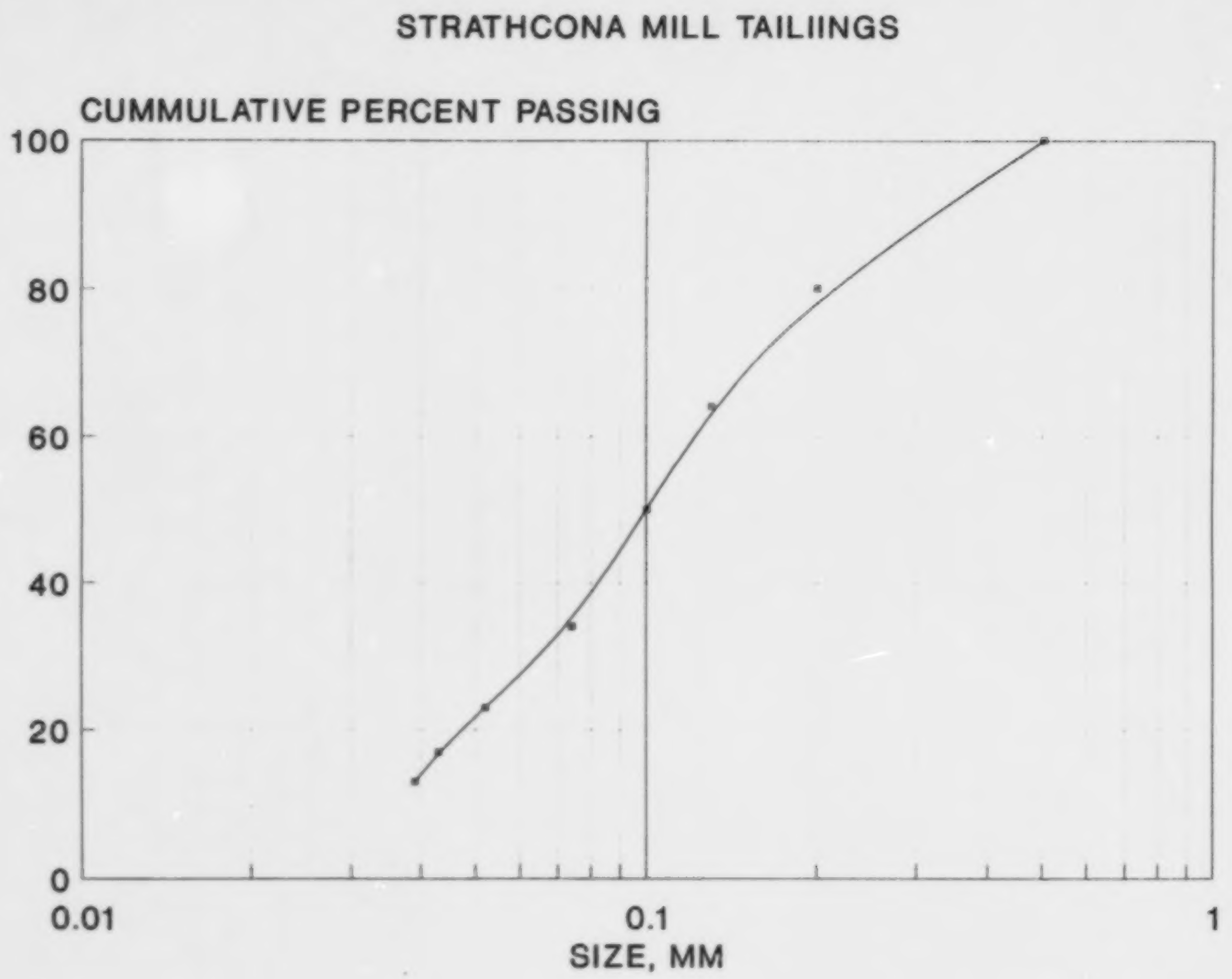


Figure 8.3, Size distribution of Strathcona mill tailings as used in hydraulic fill binder alternatives test work.

8.5.3 Initial Comparison of Binders

As an initial comparison of the binder alternatives the 28 day strength of hydraulic fill cylinders cast with each of the binders under consideration was tested. The results are shown in figure 8.4. Slag "A", with lime activation gave the highest ultimate strength, slag "B" returned the next best results and 50:50 portland and flyash the lowest strength results. The activation of the ferrous slags with lime gave an obvious improvement in fill strength development. The effect of lime activation was considered further in the next series of tests.

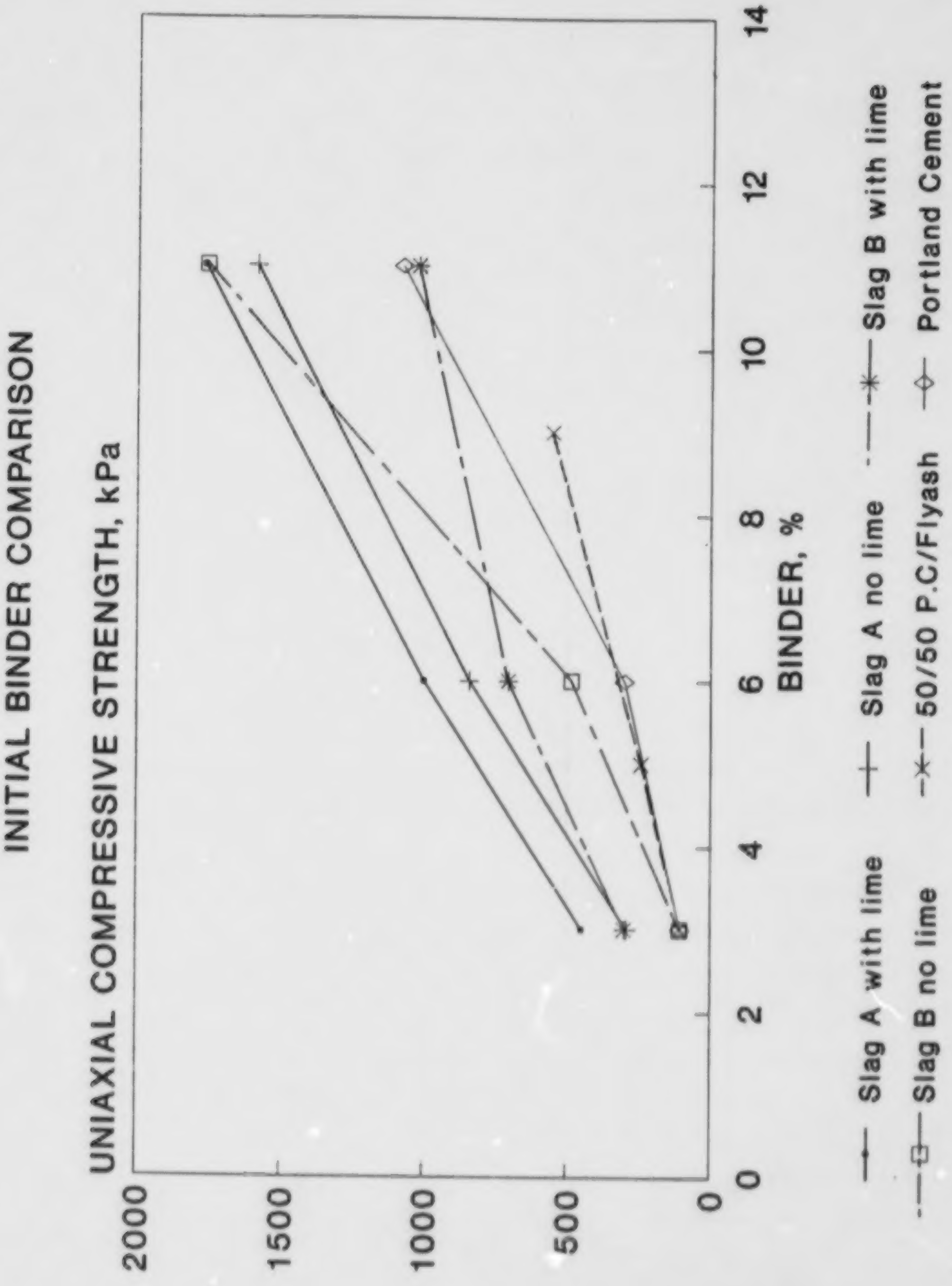
Table 8.5 presents the results of this initial test work.

			UCS strength, MPa		
% Binder addition	3%	4.6%	6%	9%	11%
Slag "A" with lime	0.45	-	1.0	-	1.8
Slag "A" without lime	0.29	-	0.84	-	1.6
Slag "B" with lime	0.30	-	0.70	-	1.0
Slag "B" without lime	0.10	-	0.48	-	1.8
50:50 portland cement type C fly ash	0.10	0.24	-	0.55	-
Portland cement	0.10	-	0.30	-	1.1

Table 8.5, UCS Strength of hydraulic tailing fill cylinders caste with various binders, MPa (binder addition by weight of tailings)

The conclusion of these initial tests was that the two ferrous slags "A" and "B" were the most attractive alternative binders. Hence the follow up work centered on how to best activate these slags.

The 50:50 flyash portland cement mixture did not perform as well as the ferrous slag binders. However it did perform well against the portland cement, essentially matching the control strengths. It is considered a viable binder alternative for hydraulic fill. The relative pricing of the binder alternatives at the time of the testing did not encourage further investigation into the flyash product, considering how well the ferrous slags seemed to be performing.



SAMPLES POURED AT 68% PULP DENSITY

Figure 8.4, Results of initial test work on hydraulic fill binder alternatives.

8.5.4 Activation of Ferrous Slags using Lime

The performance of lime as an activator for both ferrous slag "A" and "B" was tested at varying addition rates. Lime was intended to act as an activator for the slag by providing rapidly hydrated CaO and also by raising the pH of the system.

Lime was shown to have a major impact on the reactivity of the ferrous slags. Lime did not appear to be essential to the long term development of strength, but without the lime activation, strength development was very slow. A lime addition rate around 2% seemed to give the optimum results, although this varied with the tailings to binder ratio.

An optimum lime addition rate was indicated by the test work. Lime addition beyond the optimum rate was detrimental to strength development, not just neutral as expected. This has important implications for practical application of ferrous slag where excess lime addition may cause reduced fill strength.

The optimum lime addition rate in terms of a percentage of slag weight tends to vary with the tailings to binder ratio. At high tailings to binder ratios (30:1) the optimum lime addition rate appeared to be higher than at low tailing to binder ratios (8:1). It is suggested that this is a consequence of the lime addition being related to both the quantity of slag and the total mass of the system. At high tailing binder ratios (30:1) the total lime in the system (added as a percentage of binder weight) is proportionally low, so that its ability to raise pH and the concentration of hydrated CaO in the pore water will be low. Hence a higher lime addition is required to get the system to the optimum conditions for slag reactivity. At low tailing binder ratios (8:1) the total lime in the system is proportionally high, so a lower lime addition rate is required to get the system to the optimum conditions.

Why the excess addition of lime appears to be detrimental to strength development is not clear. It is an indication of the complex and subtle nature of the slag hydration reactions.

In practice the lime addition rates should vary with the tailings:binder ratio of the fill being poured. A high ratio (low strength fill) requires a high lime dosage rate (as a % of the binder weight) and a low ratio (high strength fill) requires a low dosage rate.

Table 8.6 and 8.7 present the results of these tests on both slag "A" and "B" which are also shown graphically in figures 8.5 and 8.6.

The tests showed that slag "A" performed consistently better than slag "B" when compared to the portland cement control. Figures 8.7 and 8.8 show the performance of each slag with the optimum lime addition against portland cement performance over time.

		Strength, MPa at various days of curing				
Ratio	% Lime	1 day	3 days	7 days	28 days	56 days
8:1	0	not set	not set	0.466	1.584	-
8:1	1	not set	0.139	1.111	1.769	-
8:1	2	0.072	0.293	0.733	1.113	-
8:1	3	0.049	0.259	0.536	0.872	-
P.C	Control	0.148	0.448	0.668	1.071	-
16:1	0	-	not set	not set	0.855	1.003
16:1	1	-	not set	0.095	1.027	1.286
16:1	2	-	0.043	0.537	0.960	0.950
16:1	3	-	0.099	0.331	0.648	0.778
P.C	Control	-	0.170	0.220	0.316	0.368
32:1	0	-	-	not set	0.289	0.385
32:1	1	-	-	not set	0.298	0.467
32:1	2	-	-	not set	0.439	0.525
32:1	3	-	-	0.050	0.449	0.625
P.C	Control	-	-	0.073	0.100	0.127

Table 8.6, Effect of lime activation on ferrous slag "A"

Note. P.C Control tests at the same ratio as the slag tests immediately above.
Ratio is tailings:binder by weight.

		Strength, MPa, at various days of curing				
Ratio	% Lime	1 day	3 days	7 days	28 days	56 days
8:1	0	not set	not set	0.081	1.770	-
8:1	2	not set	0.172	0.452	0.929	-
8:1	4	not set	0.097	0.370	0.855	-
8:1	6	not set	0.076	0.317	0.923	-
8:1	8	not set	0.077	0.259	1.014	-
P.C	Control	0.128	0.367	0.610	1.070	-
16:1	0	not set	not set	not set	0.483	0.972
16:1	2	not set	0.048	0.161	0.722	0.859
16:1	4	not set	0.072	0.161	0.464	0.521
16:1	6	not set	0.049	0.168	0.430	0.425
16:1	8	not set	0.026	0.121	0.416	0.415
P.C	Control	not set	0.111	0.191	0.307	0.303
32:1	0	-	not set	not set	0.101	0.254
32:1	2	-	not set	not set	0.294	0.396
32:1	4	-	not set	0.059	0.243	0.336
32:1	6	-	not set	0.070	0.212	0.265
32:1	8	-	not set	0.071	0.207	0.243
P.C	Control	-	not set	0.072	0.099	0.097

Table 8.7, Effect of lime activation on ferrous slag "B".

Note. P.C Control tests at the same ratio as the slag tests immediately above.
Ratio is tailings:binder by weight.

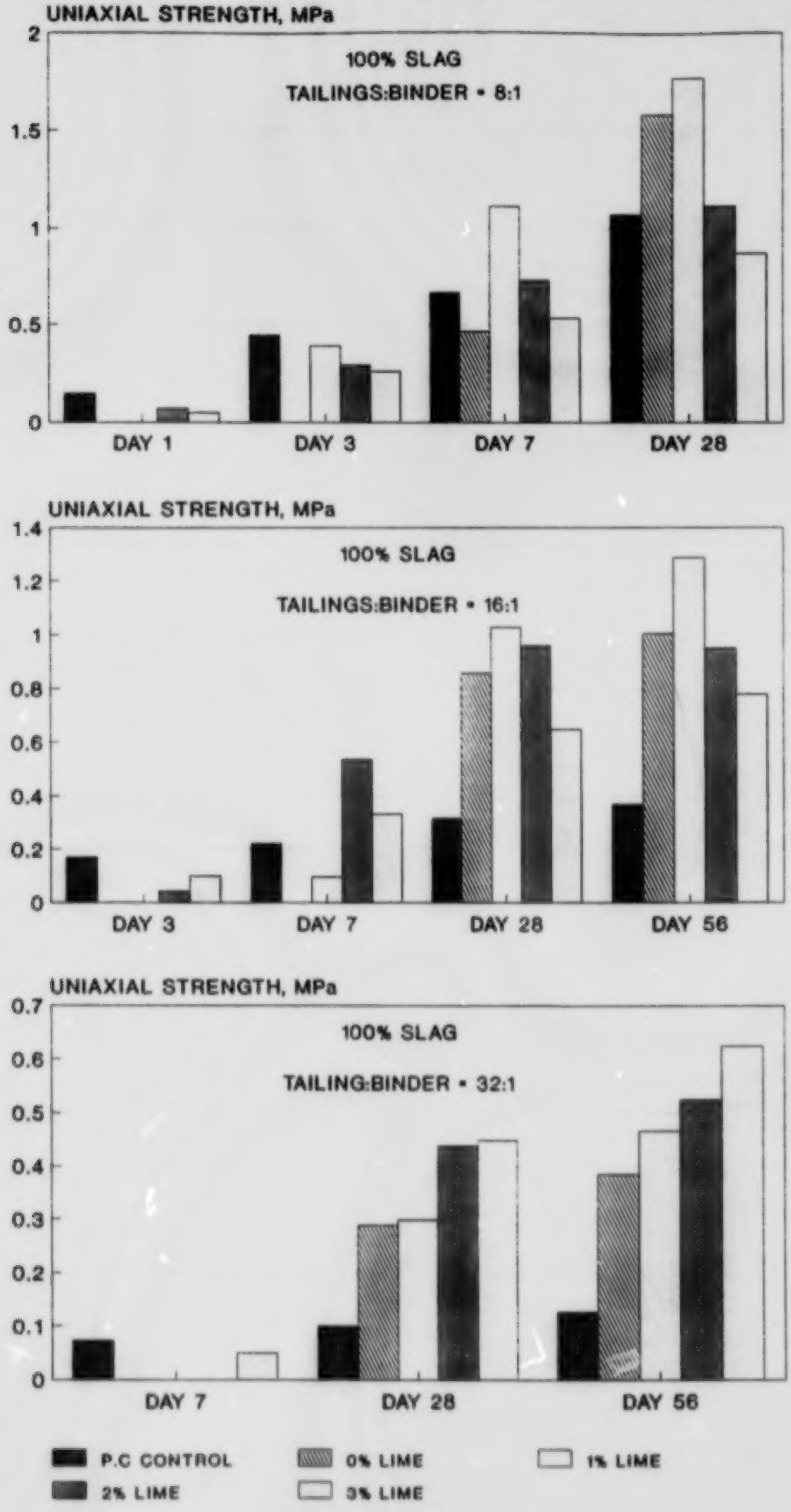
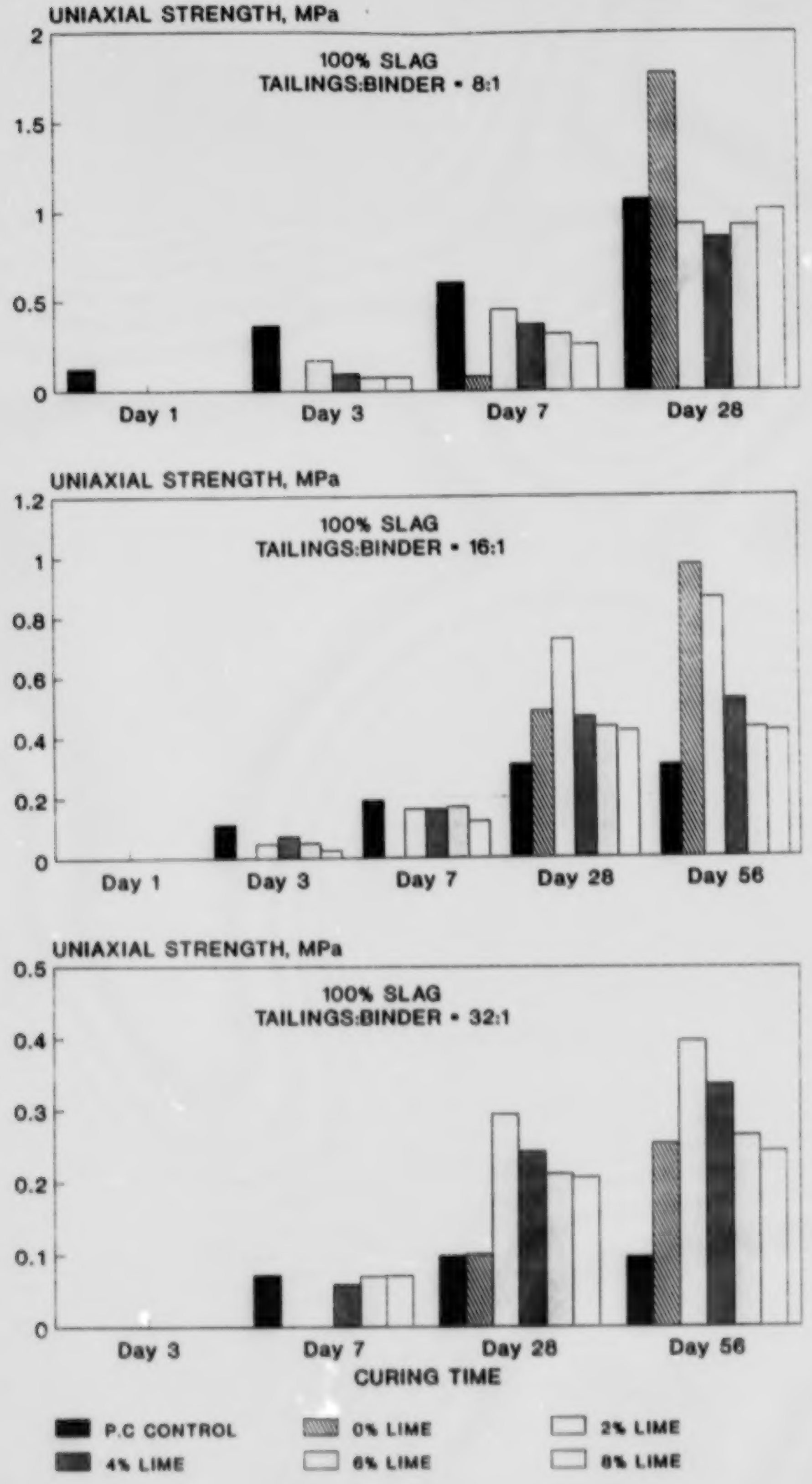


Figure 8.5, Effect of lime addition on slag "A" performance as a hydraulic fill binder.



P.C = PORTLAND CEMENT

Figure 8.6, Effect of lime addition on slag "B" performance as a hydraulic fill binder.

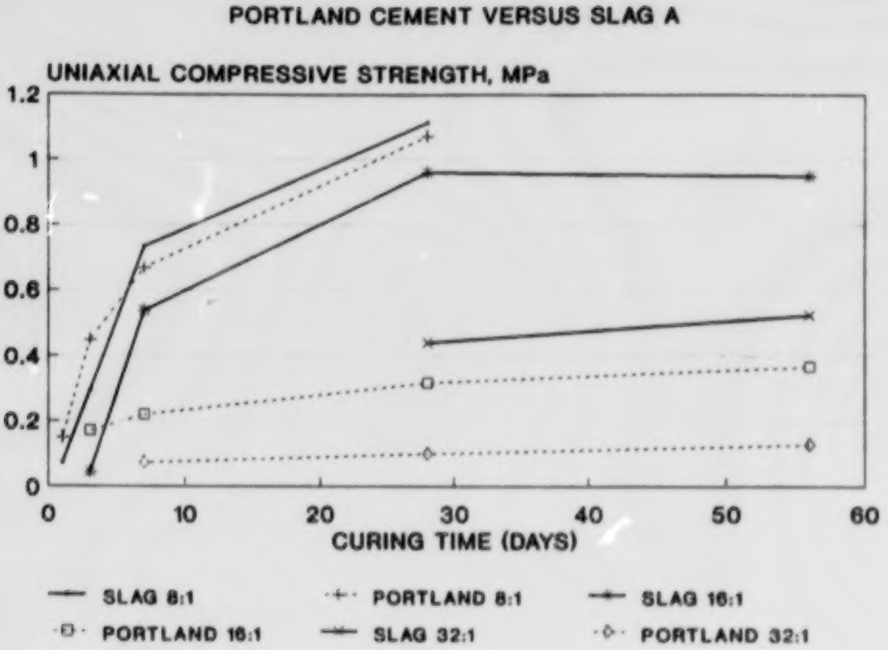


Figure 8.7, Performance of slag "A" with optimum lime addition compared to portland cement performance over time.

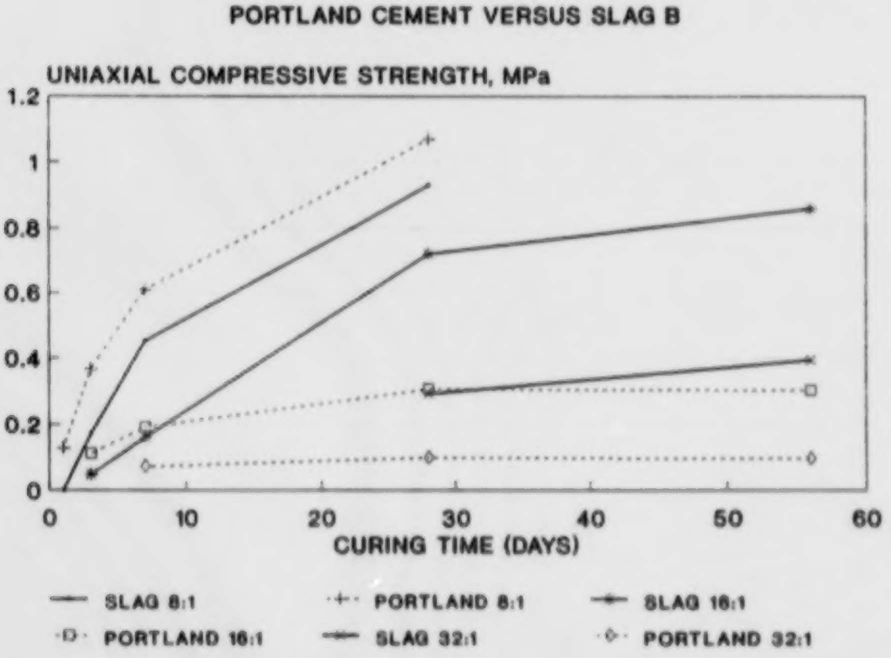


Figure 8.8, Performance of slag "B" with optimum lime addition compared to portland cement performance over time.

8.5.5 Activation of Blended Portland and Ferrous Slag with Lime

Experience from the concrete industry suggested that ferrous slags could be used to replace up to 50% of the portland cement in a pour without loss of strength, but with some reduction in the curing rate. The results from the use of 100% slag with lime activation described above were remarkable and in conflict to the 50% limit for concrete technology. A series of tests was carried out to determine the performance of blended portland cement and ferrous slag mixtures in the ratio 50:50 and 25:75. In addition lime was added at varying rates to determine if its beneficial activation effects noticed with 100% slag would be repeated with portland cement present.

Lime addition was intended to provide an additional source of hydrated CaO for the rapid activation of the ferrous slag. Lime addition rates of 0 to 3% by weight of the combined portland and slag were tested. Performance of the activated binders was tested by measuring the uniaxial strength of cast cylinders of tailings fill.

Lime addition made no difference to the strength achieved with blended portland cement and slag. It is proposed that the portland cement components of the blended binders is sufficient to provide enough hydrated CaO and high enough pH that the addition of lime had no influence. Lime is added to provide hydrated CaO and pH, but with portland cement present the system is already saturated in these components.

The blended portland cement and ferrous slag mixtures did not perform as well as the 100% slag with lime activation. It is suggested that because slag was such a good binder with Strathcona tailings that the portland cement simply acted as a weaker substitute.

The results of the test work are presented in tables 8.8 to 8.11 and graphically in figures 8.9 to 8.12.

		Strength, MPa, at various days of curing				
Ratio	% Lime	1 day	3 days	7 days	28 days	56 days
8:1	0	0.90	0.247	0.544	1.621	-
8:1	1	0.078	0.220	0.581	1.336	-
8:1	2	0.082	0.211	0.514	1.347	-
8:1	3	0.076	0.200	0.519	1.428	-
P.C	Control	0.148	0.448	0.668	1.071	-
16:1	0	-	0.094	0.169	0.501	0.594
16:1	1	-	0.079	0.178	0.519	0.592
16:1	2	-	0.072	0.181	0.497	0.575
16:1	3	-	0.071	0.170	0.518	0.599
P.C	Control	-	0.170	0.220	0.316	0.368
32:1	0	-	-	0.057	0.173	0.225
32:1	1	-	-	0.059	0.170	0.228
32:1	2	-	-	0.057	0.170	0.228
32:1	3	-	-	0.057	0.169	0.233
P.C	Control	-	-	0.073	0.100	0.127

Table 8.8, Effect of lime addition to 50:50 ferrous slag "A" : portland cement

		Strength, MPa, at various days of curing				
Ratio	% Lime	1 day	3 days	7 days	28 days	56 days
8:1	0	0.048	0.136	0.463	1.380	-
8:1	1	0.050	0.125	0.471	1.374	-
8:1	2	0.044	0.122	0.506	1.495	-
8:1	3	0.043	0.112	0.423	1.391	-
P.C	Control	0.148	0.448	0.668	1.071	-
16:1	0	-	0.056	0.193	0.607	0.749
16:1	1	-	0.054	0.207	0.617	0.736
16:1	2	-	0.054	0.199	0.617	0.747
16:1	3	-	0.052	0.201	0.601	0.742
P.C	Control	-	0.170	0.220	0.316	0.368
32:1	0	-	-	0.164	0.337	0.387
32:1	1	-	-	0.154	0.359	0.379
32:1	2	-	-	0.148	0.390	0.427
32:1	3	-	-	0.132	0.341	0.381
P.C	Control	-	-	0.073	0.100	0.127

Table 8.9, Effect of lime addition on 75:25 ferrous slag "A" : portland cement

		Strength, MPa, at various days of curing				
Ratio	% Lime	1 day	3 days	7 days	28 days	56 days
8:1	0	0.052	0.154	0.346	0.859	-
8:1	2	0.051	0.152	0.332	0.908	-
8:1	4	0.046	0.158	0.354	0.921	-
8:1	6	0.048	0.145	0.352	0.949	-
8:1	8	0.034	0.161	0.356	0.975	-
P.C	Control	0.128	0.367	0.610	1.070	-
16:1	0	not set	0.064	0.100	0.310	0.415
16:1	2	not set	0.066	0.101	0.303	0.423
16:1	4	not set	0.061	0.101	0.321	0.375
16:1	6	not set	0.068	0.102	0.316	0.407
16:1	8	not set	0.067	0.119	0.317	0.405
P.C	Control	not set	0.111	0.191	0.307	0.303
32:1	0	-	not set	0.050	0.117	0.118
32:1	2	-	not set	0.049	0.117	0.137
32:1	4	-	not set	0.045	0.112	0.128
32:1	6	-	not set	0.048	0.102	0.125
32:1	8	-	not set	0.048	0.101	0.110
P.C	Control	-	not set	0.072	0.099	0.097

Table 8.10, Effect of lime addition on 50:50 ferrous slag "B" : portland cement.

Ratio	% Lime	Strength, MPa, at various days of curing 1 day	3 days	7 days	28 days	56 days
8:1	0	0.028	0.112	0.272	0.906	-
8:1	2	0.033	0.105	0.269	0.925	-
8:1	4	0.027	0.096	0.285	0.985	-
8:1	6	0.026	0.103	0.290	0.937	-
8:1	8	not set	0.085	0.287	0.990	-
P.C	Control	0.128	0.367	0.610	1.070	-
16:1	0	not set	0.046	0.085	0.387	0.513
16:1	2	not set	0.046	0.094	0.356	0.459
16:1	4	not set	0.041	0.098	0.360	0.499
16:1	6	not set	0.044	0.101	0.363	0.466
16:1	8	not set	0.045	0.100	0.341	0.459
P.C	Control	not set	0.111	0.191	0.307	0.303
32:1	0	-	not set	0.058	0.121	0.268
32:1	2	-	not set	0.049	0.144	0.250
32:1	4	-	not set	0.047	0.144	0.245
32:1	6	-	not set	0.050	0.161	0.233
32:1	8	-	not set	0.043	0.141	0.224

Table 8.11, Effect of lime addition on 25:75 ferrous slag "B" portland cement.

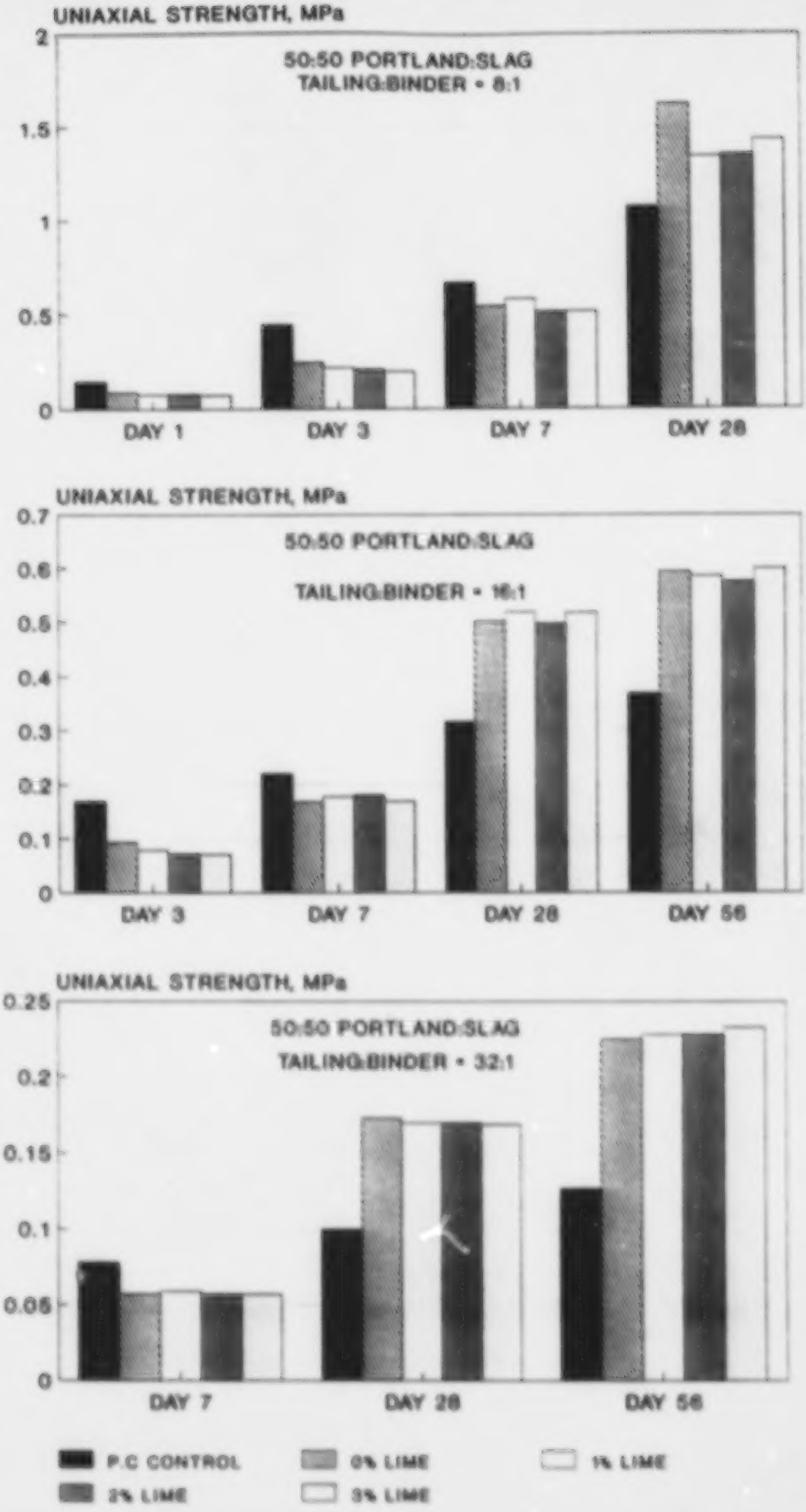


Figure 8.9, Effect of lime addition on blended 50:50 slag"A":portland cement.

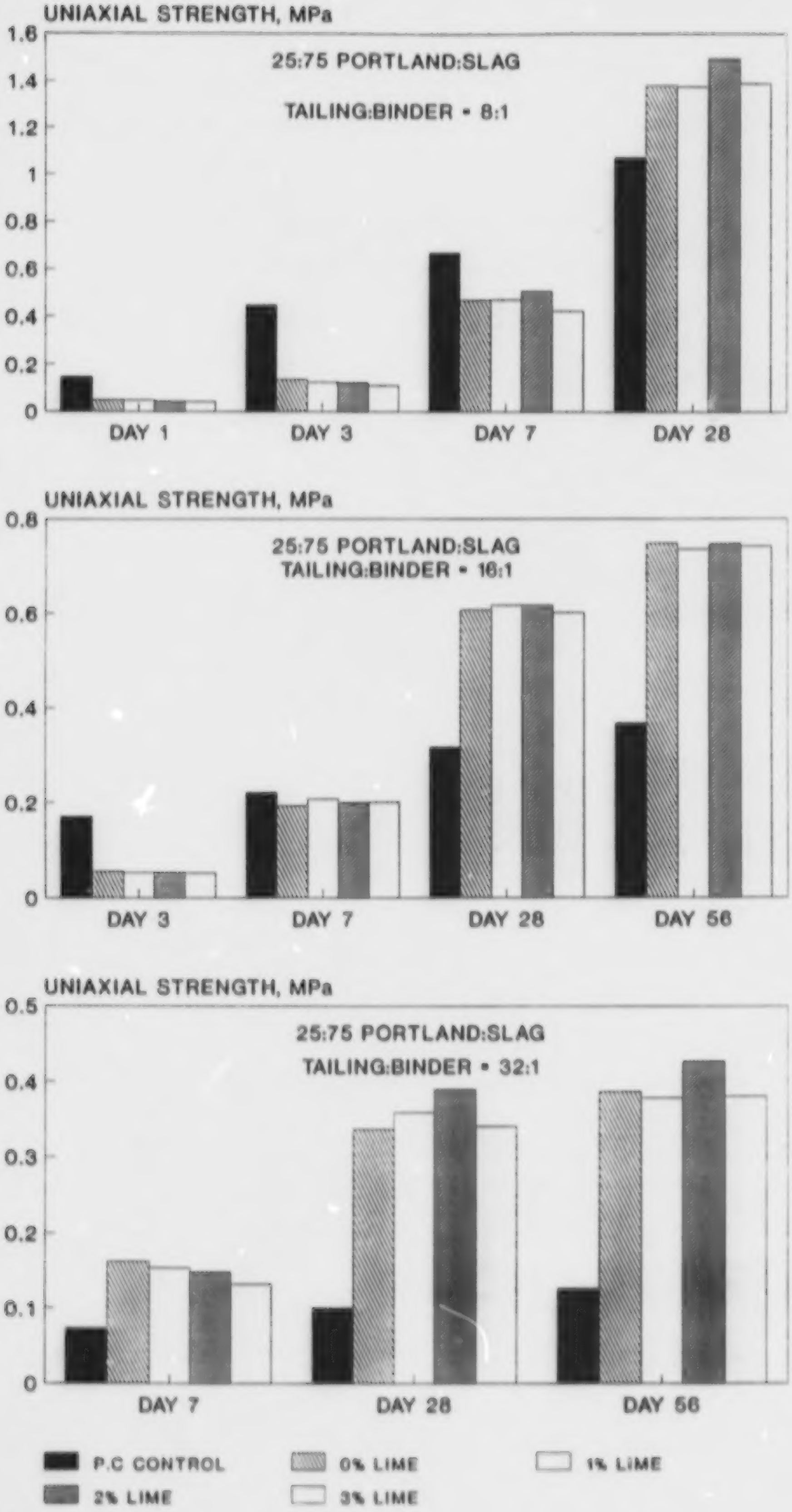


Figure 8.10, Effect of lime addition on blended 75:25 slag "A":portland cement.

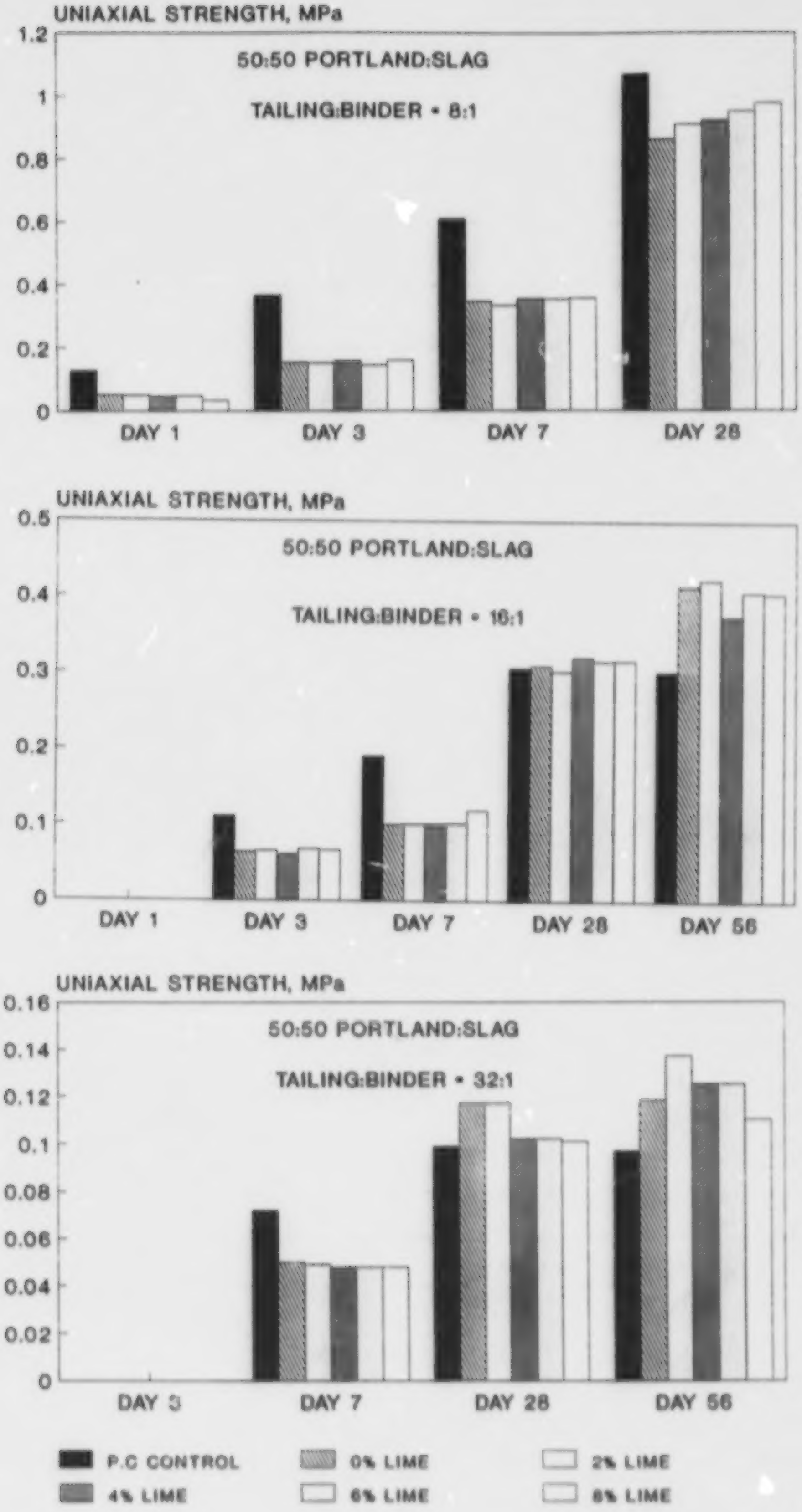
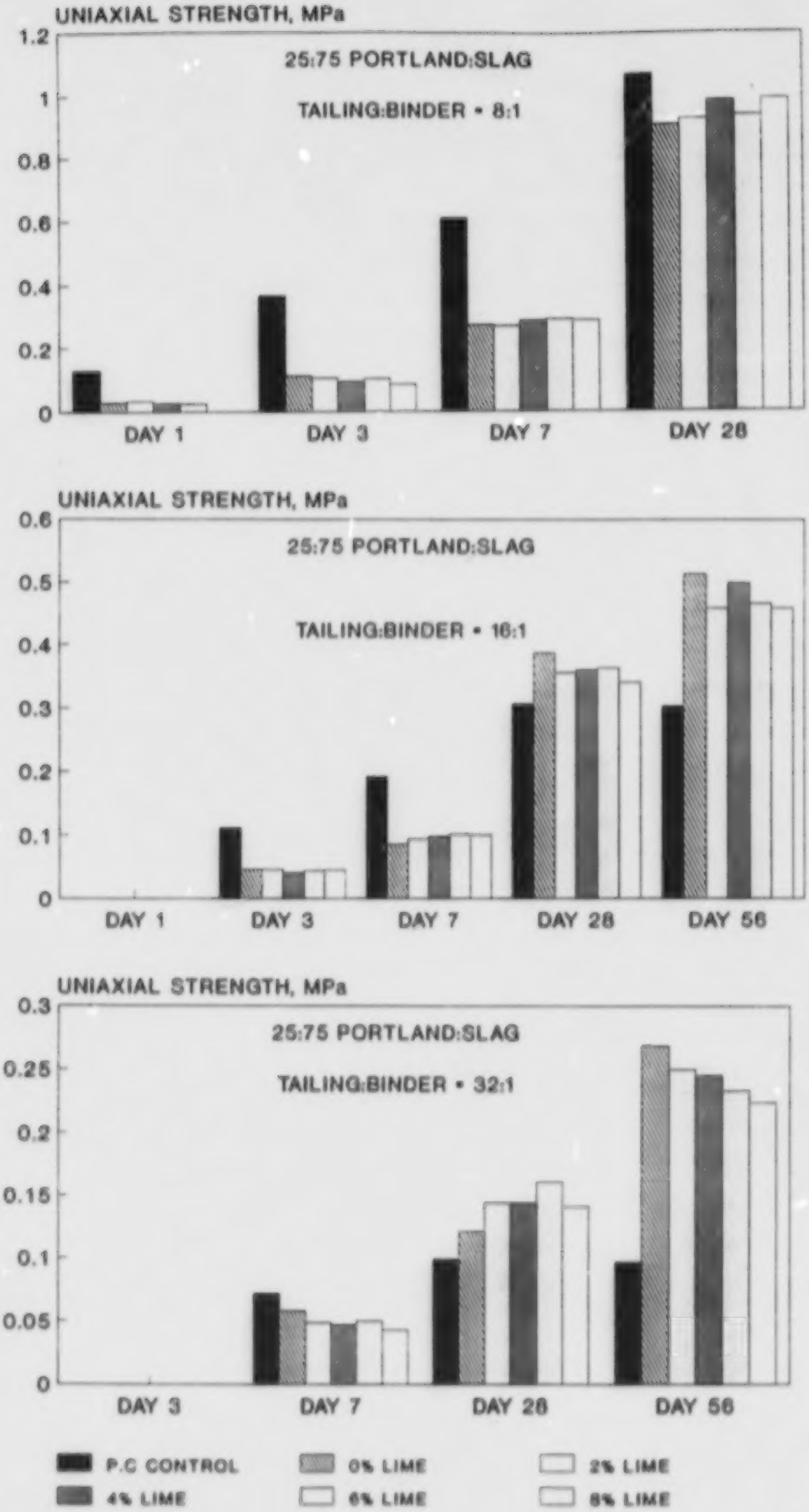


Figure 8.11, Effect of lime addition on blended 50:50 slag "B":portland cement.



P.C • PORTLAND CEMENT

Figure 8.12, Effect of lime addition on blended 75:25 slag "B":portland cement.

8.5.6 Chemical Interaction

The strengths returned by the slag with lime activation were superior to those obtained with blended portland and slag. This is slightly in contradiction to experience in concrete technology, where it is generally accepted that a ferrous slag can replace up to about 50% of the portland cement before strength begins to be adversely affected. The same limit to portland replacement was expected with the use of ferrous slags. It is suggested that there is some unknown chemical interaction occurring between the minerals in the tailing material and the slag, which assists the slag hydration. This suggests that the slags tested in this project may not perform as well with different tailings materials, alluvial sands or rock fills as it has with Strathcona mill tailings. This is another example of the subtle nature of the binder hydration chemistry.

8.5.7 Underground Field Test

Laboratory tests are the most common method of comparing the various backfill binders. Because under ground conditions vary from those in the lab a full scale field trial is mandatory to confirm that any improvements indicated from a lab study are obtained underground.

To evaluate slag "A" as a binder alternative a typical cut and fill stope on the 2700 level of Falconbridge's Strathcona Mine was chosen for a field trial. This ore zone is steeply dipping (approximately 70 degrees), roughly 4.5 m wide and 36.5 m in strike length. Two layers of hydraulic fill were poured using slag "A" as a binder. Each layer contained a different binder addition rate. The first, or bottom 1 m was filled with backfill containing 3% (32:1 tailings:binder) slag "A" and the top or final 1 m was filled with 6% binder addition. Figure 8.13 shows the trial layout.

The two layers were poured with one day between pours. Each pour was observed by the mine operating personnel for their subjective comments. A pressure meter cell was installed at the bottom of the fill fence to monitor the hydraulic head resulting from hydraulic filiing. A low measurement of pressure would indicate effective excess fill water drainage. Diamond drilling of this fill to obtain core samples was considered, but was deemed to be too costly.

The pouring of both layers of the test fill was observed by a mine captain, shift supervisor and the backfill technology engineer. No special design changes were made to the backfill plant on surface or to the stope to be filled underground, in order to complete the trial. During the first test pour (3% slag) the mine

Captain was able to walk on the fill within half a metre of the backfill discharge point and commented, "I have not seen backfill of this quality in many years".

After one month of curing, technical personnel returned to the test site to extract a large test sample. A sample was to be taken from the more highly consolidated top layer. However, hand tools could not break away a reasonable sample of the fill and sampling efforts were discontinued.

The results of the pressure cell located at the base of the fill fence showed that there was no hydraulic pressure build up during or after backfilling. This fact and the ability of men to walk directly beside the backfill pour point during the pour, something that is not common for our consolidated fill, indicated good fill drainage.

No underground design changes appear necessary in order to use slag "A" as a binder.

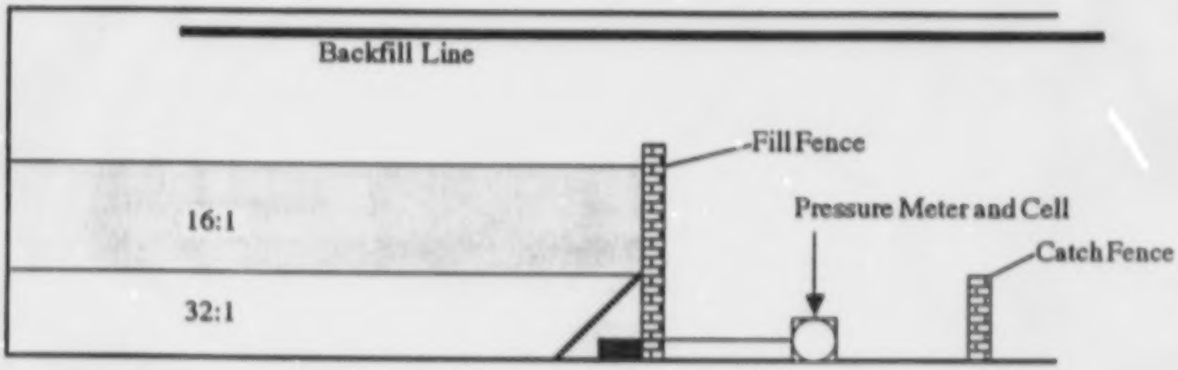


Figure 8.13, Layout of underground field trial of slag"A".

8.5.8 Economic Comparison

Slag showed that it could not only reduce the cost of a mine's consolidated backfill, but can also improve the fill quality. An example of this is shown when mill tailings from Falconbridge's Strathcona mill are consolidated with 5% slag cement, by weight, rather than 5% portland cement. The savings in an example like this can be as high as $3.41 per tonne of fill poured, or a 64% binder cost saving over the same fill consolidated with portland cement. The fill that is consolidated with slag can be as strong as the fill with the Portland cement after only four days and up to 350% stronger after one month of curing.

The potential economic advantages of using slag "A" as a hydraulic fill binder were estimated by considering a typical blast hole stope application. The two advantages of using slag are;

1. Slag can be purchased at a considerable price discount over portland cement, (on an equal tonnage basis)

2. Slag has the potential to provide greater strength at equivalent addition rates, so less slag may be required to achieve design strengths.

For the purposes of the economic comparison the price used for each binder was; portland cement $105/ton, slag "A" $75/ton. These are not contract prices, but are approximate costs for these materials in Northern Ontario (1988).

A typical blasthole stope at Falconbridge's Lockerby Mine in Sudbury was used to make the economic comparison. This stope has dimensions 175 feet high, 36 feet wide and 50 feet deep and requires 2250 tons of consolidated fill, 6.6% (14:1) binder, as a plug. The remaining 13,500 tons of fill contain 4.8% (20:1) binder and constitutes the bulk of the pour. These are common consolidation ratios for a stope of this type that will encounter mining on both sides. Backfill consolidation is designed to minimize fill failure during the subsequent adjacent mining operations and thereby reduce dilution. Figure 8.8 shows the stope geometry.

Figure 8.14 presents the results of the economic comparisons. Three options are compared;

1. Only portland cement as a binder.

2. Only slag "A" as a binder. No reduction in total binder consumption.

3. Only slag "A" as a binder. Half the total binder quantity of binder used, i.e. slag added at half the rate at which portland cement would be added.

Case 3 is considered to be conservative because data indicates that, as a result of the much higher 28 day strength of slag over portland cement (up to 4 times), considerably less slag could be used and still equal the 28 day strength of portland.

Binder Type	Fill Zone	Ratio fill:binder	28-day Strength (PSI)	Binder Tons (ton)	Total Tons (ton)	Total Cost $ Can.	Savings $ Can.
Portland	Plug	14:1	58	150			
cement	Stope	20:1	35	643	793	83,265	
Ferrous slag,	Plug	14:1	170	150			
full replacement	Stope	20:1	105	643	793	59,475	23,790
Ferrous slag,	Plug	28:1	83	78			
half replacement	Stope	40:1	48	322	400	30,000	53,265

Table 8.12, Binder ratios, costs and savings

STOPE	BINDER TONS	PORTLAND CEMENT		FERROUS SLAG (full replacement)		FERROUS SLAG (half replacement)	
		Cost ($)	Strength (PSI)	Cost ($)	Strength (PSI)	Cost ($)	Strength (PSI)
Plug pour	150	15,750	58	11,250	170	5,625	83
Bulk pour	643	67,515	35	48,225	105	24,113	48
Total	793	83,265		59,475		29,738	

Table 8.13, Binder costs and 28 day strengths

BINDER	BINDER COST / TON FILL ($ / ton)	SAVINGS OVER PORTLAND CEMENT ($ / ton fill)	($ / Stope)
Portland Cement	5.30	---	
Ferrous Slag (full replacement)	3.79	1.51 (28 %)	23,850
Ferrous Slag (half replacement)	1.89	3.41 (64 %)	53,662

Note: Savings are for a fill with 5 % binder added

Table 8.14, Binder cost per ton of fill and savings

Both laboratory and field tests indicate that there are considerable cost savings and quality improvements possible to the hydraulic tailings backfill when slag cement "A", activated with lime, is used as a binder in place of portland cement. The binder cost savings can be in excess of 65% and fill strength improvement of up to 350% is possible.

Figure 8.15 shows where pozzolans are produced in Canada.

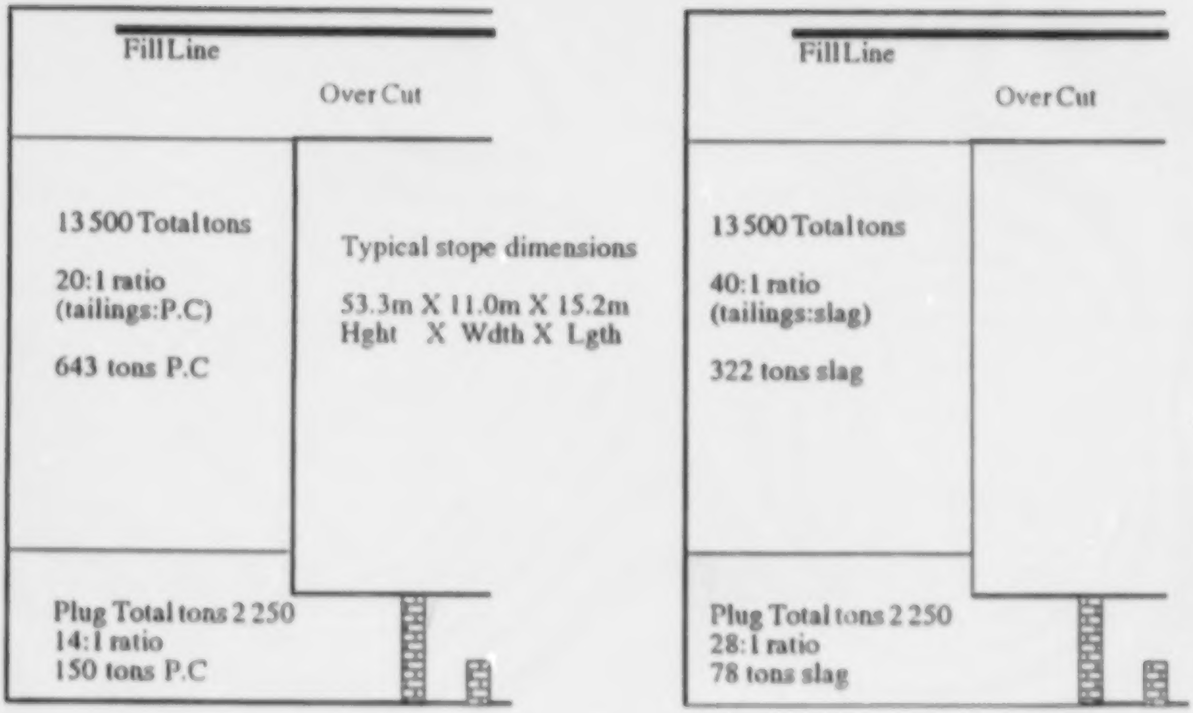

Fill Line
Over Cut
13 500 Total tons
20:1 ratio
(tailings:P.C)
643 tons P.C
Typical stope dimensions
53.3m X 11.0m X 15.2m
Hght X Wdth X Lgth
Plug Total tons 2 250
14:1 ratio
150 tons P.C
Fill Line
Over Cut
13 500 Total tons
40:1 ratio
(tailings:slag)
322 tons slag
Plug Total tons 2 250
28:1 ratio
78 tons slag

Figure 8.14, Economic comparison of slag "A" against normal portland cement.

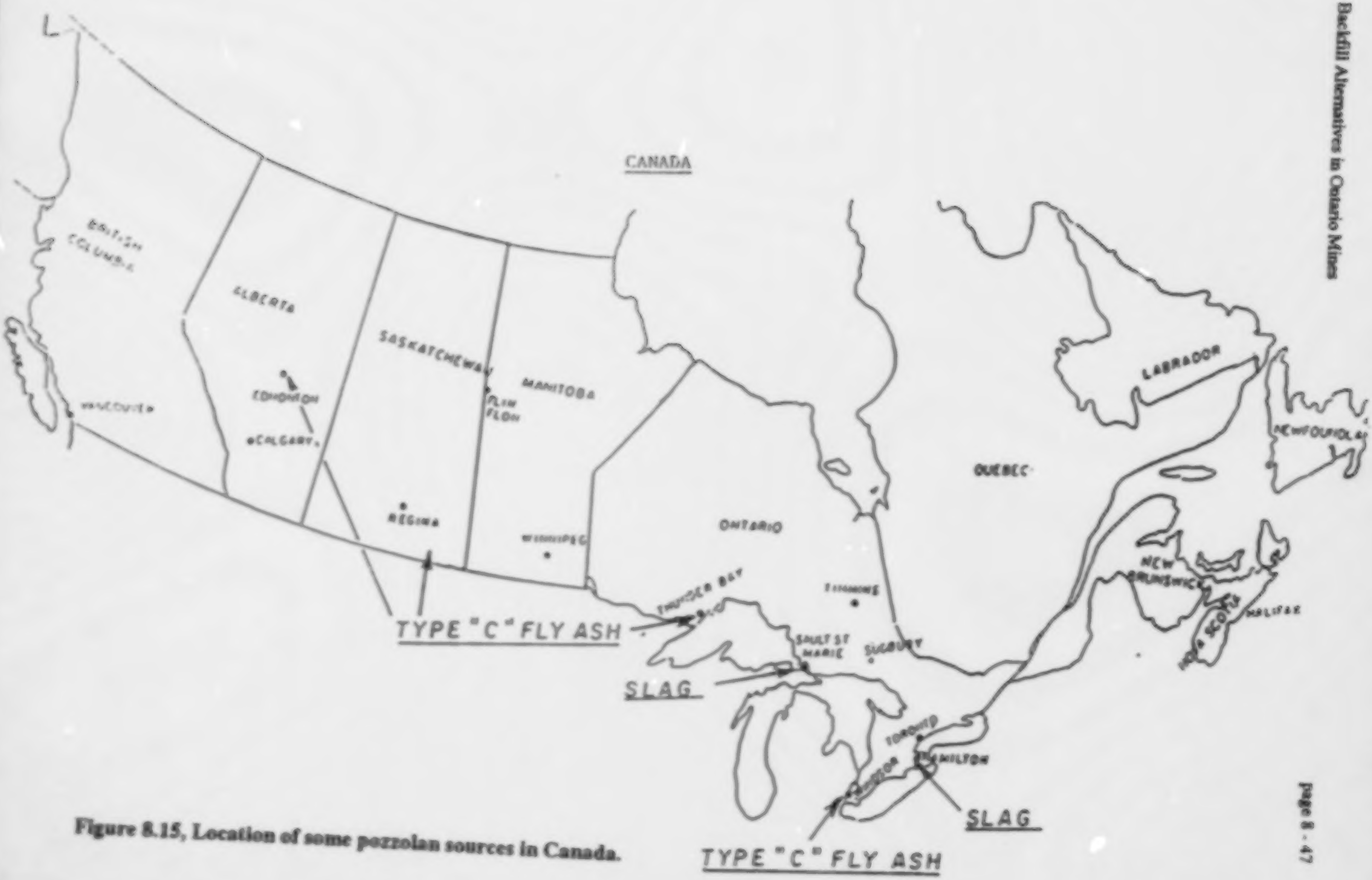

CANADA
BRITISH COLUMBIA
VANCOUVER
ALBERTA
EDMONTON
CALGARY
SASKATCHEWAN
REGINA
MANITOBA
FLIN FLON
WINNIPEG
ONTARIO
THUNDER BAY
TIMMINS
SAULT ST MARIE
SUDBURY
TORONTO
HAMILTON
QUEBEC
LABRADOR
NEWFOUNDLAND
NEW BRUNSWICK
NOVA SCOTIA
HALIFAX
TYPE "C" FLY ASH
SLAG
SLAG
TYPE "C" FLY ASH

Figure 8.15, Location of some pozzolan sources in Canada.

8.6 BINDER ALTERNATIVES FOR ROCK FILL, RESEARCH WORK BY KIDD CREEK MINES LIMITED

Kidd Creek carried out a comprehensive testing program as part of this project to evaluate the cementitious properties of different binders on rockfill with the addition of commercially available chemicals.

The large number of tests to be conducted meant that it would not be practical to use full size rockfill test cylinders which are typically 0.45m by 0.91m in dimension and require a lot of material. These are used to make sure a representative sample of rock sizes is obtained. However they are difficult and costly to work with. For this comparative study of binder alternatives it was considered valid to use a standard aggregate mix sized to allow the use of smaller laboratory sized test cylinders. The aggregate used in the cylinder tests described here was a mixture of washed gravel and alluvial sand at a ratio of 2 to 1. The maximum aggregate size was 1.8 cm. All the cylinder tests were carried out using 7.35 cm diameter, 14.7 cm long, cardboard moulds. Each mix contained 5% by weight of cementitious material. A water cement ratio of 1.2 by weight was maintained in all sets unless mentioned otherwise. To reduce the effect of variation in the composition of portland cement on the test strength, a blend of normal Type 10 portland cement from four cement suppliers was used in the program. A minimum of three cylinders per set were tested.

8.6.1 Test Work on Kidd Creek Copper Slag as a Binder Alternative

8.6.1.1 Test the reactivity of copper slag

A simple series of cube tests was conducted to determine if Kidd Creek Mine's copper slag showed any pozzolanic activity and to investigate the rate of strength development (report 2, appendix A). Finely ground slag (at 4000 Blaine size) was added to portland cement and used as a binder to caste 2 inch cubes of rockfill. The compressive strength of these cubes was tested after 28 days of curing and compared to control cubes containing only portland cement as a binder.

The results indicated that ground copper slag did possess moderate pozzolanic activity in the presence of portland cement and its addition as a binder would improve the strength of rockfill.The results suggest that a mix of 60% portland cement with 40% ground slag would make an acceptable economic binder capable of exceeding the performance of straight portland cement if long curing periods are provided.

1. With slag replacing 40% of the portland cement the rockfill attained 92% of the control strength, and since the slag curing process is much slower than that of portland cement, the long term strength achieved with such a blend should surpass that achieved with straight portland cement.

2. With slag replacing 60% of the portland cement the rockfill was significantly weaker than the control (45% less). Slag replacement should not exceed 40%.

3. By comparing the strength of rockfill with constant addition of portland cement with and without addition of slag the test showed that slag made a major contribution to strength development. High water cement and sand cement ratios in the samples without slag addition may have contributed to the low compressive strengths measured.

Binder Composition (total binder addition: 37% of aggregate weight)	Strength Achieved (28 day curing)
100% portland cement (control)	22.4 MPa
60% portland cement & 40% ground slag	20.7 MPa
40% portland cement & 60% ground slag	14.2 MPa
40% portland cement	4.6 MPa
60% portland cement	11.7 MPa

Table 8.15, Compressive strength of cube samples.

Note: 40% portland cement means that the samples were bound with 40% of the weight of portland used in the control samples, similarly for 60% portland cement.

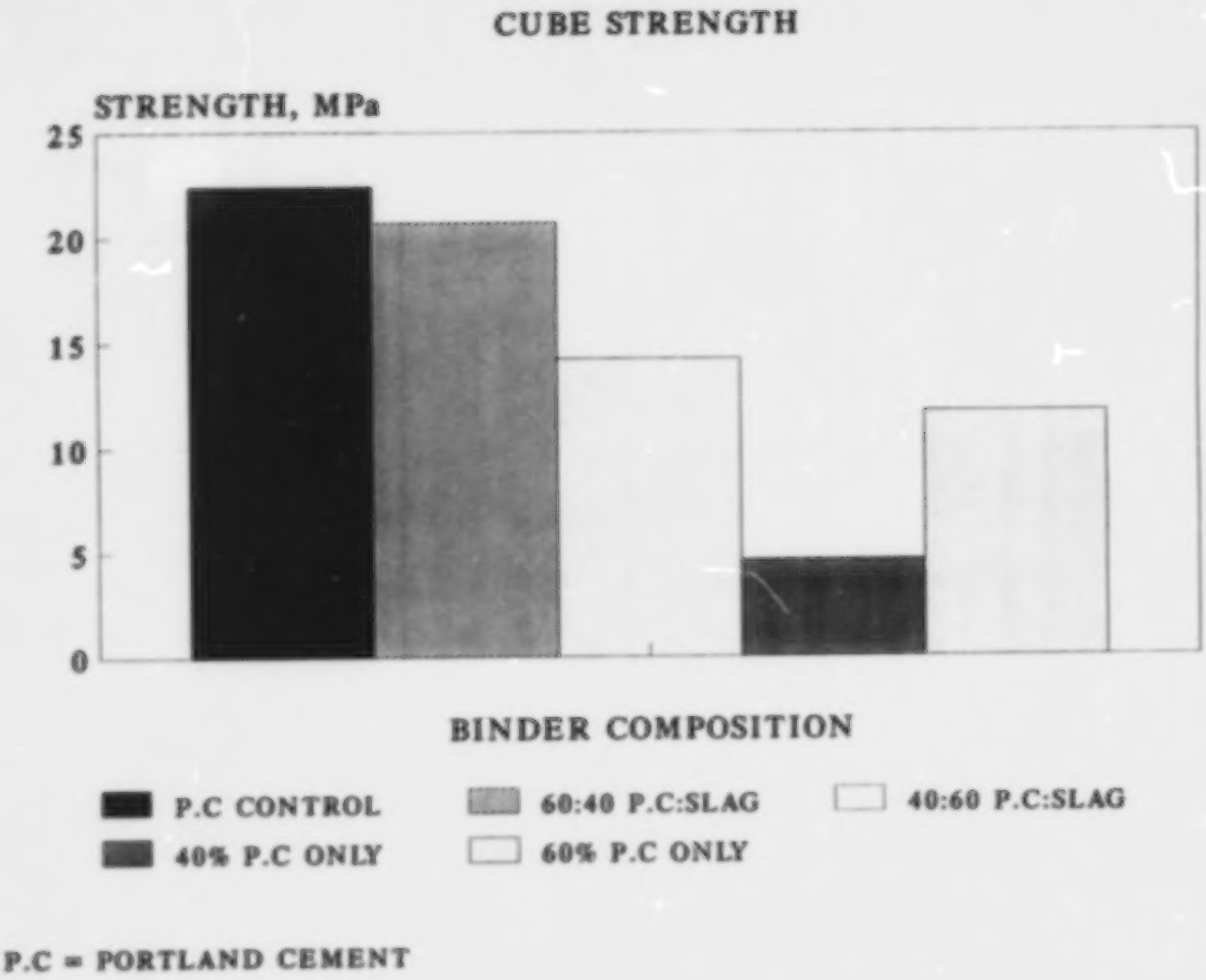


Figure 8.16, Performance of copper slag as a rockfill binder compared to portland cement.

8.6.1.2 Effect of fineness of grind on performance

The affect of fineness of grind of Kidd Creek Mine's copper slag on the compressive strength development of rockfill was investigated (report 1, appendix A). Results indicate that after 3000 Blaine size there is little difference in performance with increasing fineness. This is in agreement with other work which suggests diminishing increase in reactivity of slag once a certain Blaine size has been reached (Churcher, 1990).

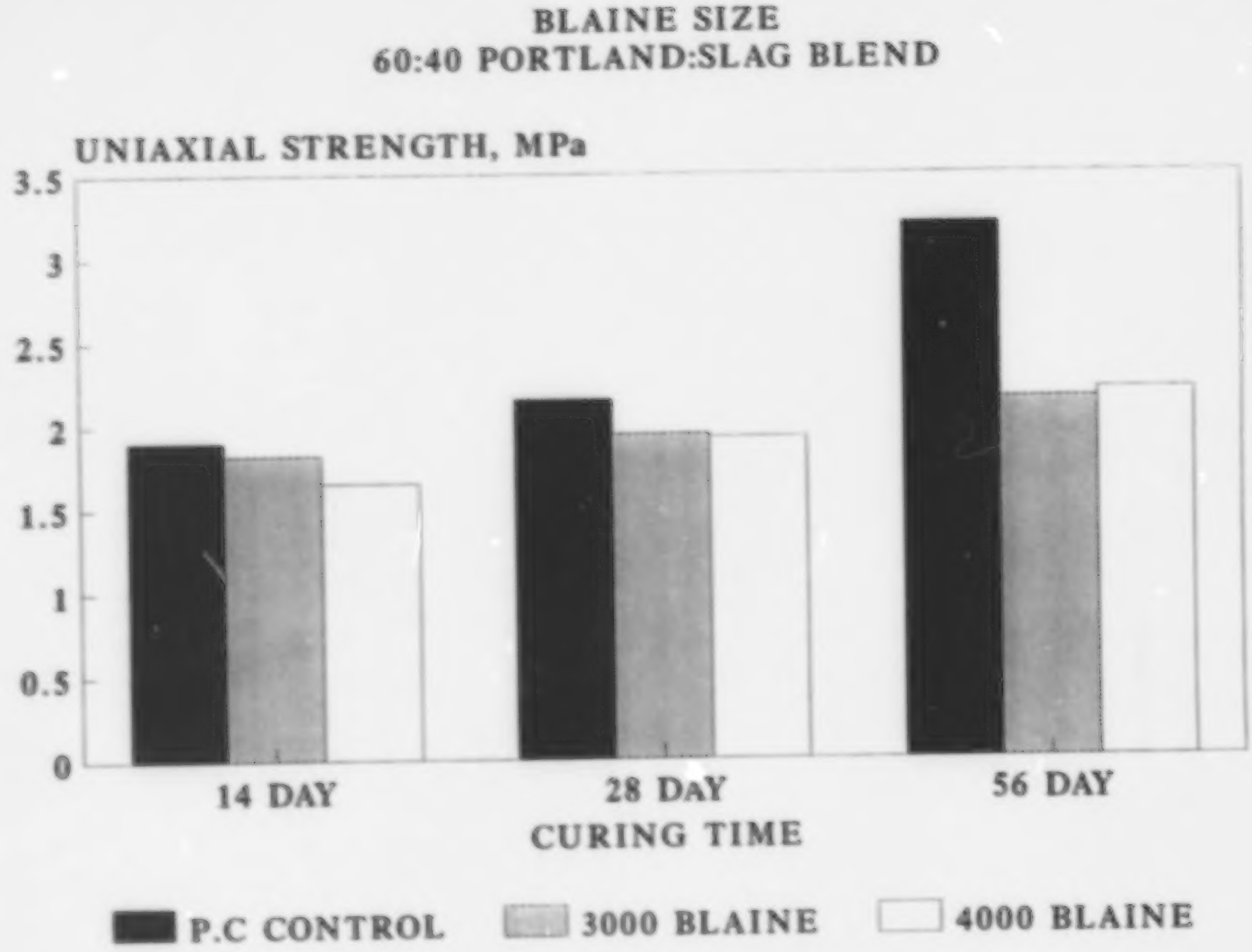
A mixture of 60% portland cement and 40% slag at a total binder content of 5% by weight of aggregate was used for these tests. The slag used was Kidd Creek Mines regular copper slag which had a Work Index value of 28.9, and D_{80} of 1960. Figure 8.17 shows the results.

1. The slag at 3000 Blaine gave encouraging results at 14 and 28 days (90 and 95% of control strength) but less so after 56 days of curing (68% of control strength).

2. Increasing the fineness of the slag from 3000 to 4000 Blaine did not improve the performance of the slag as a binder.

Binder Composition (5% binder by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 14 day	28 day	56 day
100% portland cement (control)	1.90	2.15	3.20
60% portland cement & 40% slag @ 3000 Blaine	1.82	1.94	2.15
60% portland cement & 40% slag @ 4000 Blaine	1.65	1.92	2.20

Table 8.16, Compressive strength of rockfill with slag at different Blaine size.



P.C = PORTLAND CEMENT

Figure 8.17, Effect of Blaine size on copper slag performance as a rockfill binder.

8.6.1.3 Effect of requenching and lime enrichment

The performance of Kidd Creek Mine's copper slag in a requenched form with varying amounts of lime enrichment was compared to un-requenched slag and portland cement (report 1, appendix A). Slag was remelted and quenched in a lime solution ($Ca(OH)_2$) in an attempt to make the chemical composition of the slag closer to that of portland cement. An improvement in the binding ability of the slag was expected as the amount of lime enrichment increased.

Requenched slag with a high lime enrichment gave better long term strength development than un-requenched slag, but not a significant improvement. Results with low lime enrichment were not so good. The affect of lime addition as a separate powdered phase was studied in a another set of tests.

From the first tests a mixture of 60% portland cement with 40% ground copper slag as a binder mixture seemed to be a reasonable composition which might be used in practice. Hence this was the binder mix used for these tests. A total binder content of 5% by weight of aggregate is typical for rockfilling operations and was used for these tests.

1. High lime requenched slag gave the best results of the slag specimens tested, achieving 75% of the control strength at 56 days curing.

Binder Composition	Strength, MPa, Achieved at Various Curing Times		
(5% binder by weight of aggregate)	14 day	28 day	56 day
100% portland cement (control)	1.90	2.15	3.20
Un-requenched slag @ 3000 Blaine (control)	1.82	1.94	2.15
Requenched slag @ 3050 Blaine, high lime	1.75	2.00	2.40
Requenched slag @ 3214 Blaine, low lime	1.56	1.70	2.00

Table 8.17,Compressive strength of rockfill with requenched slag as a binder.

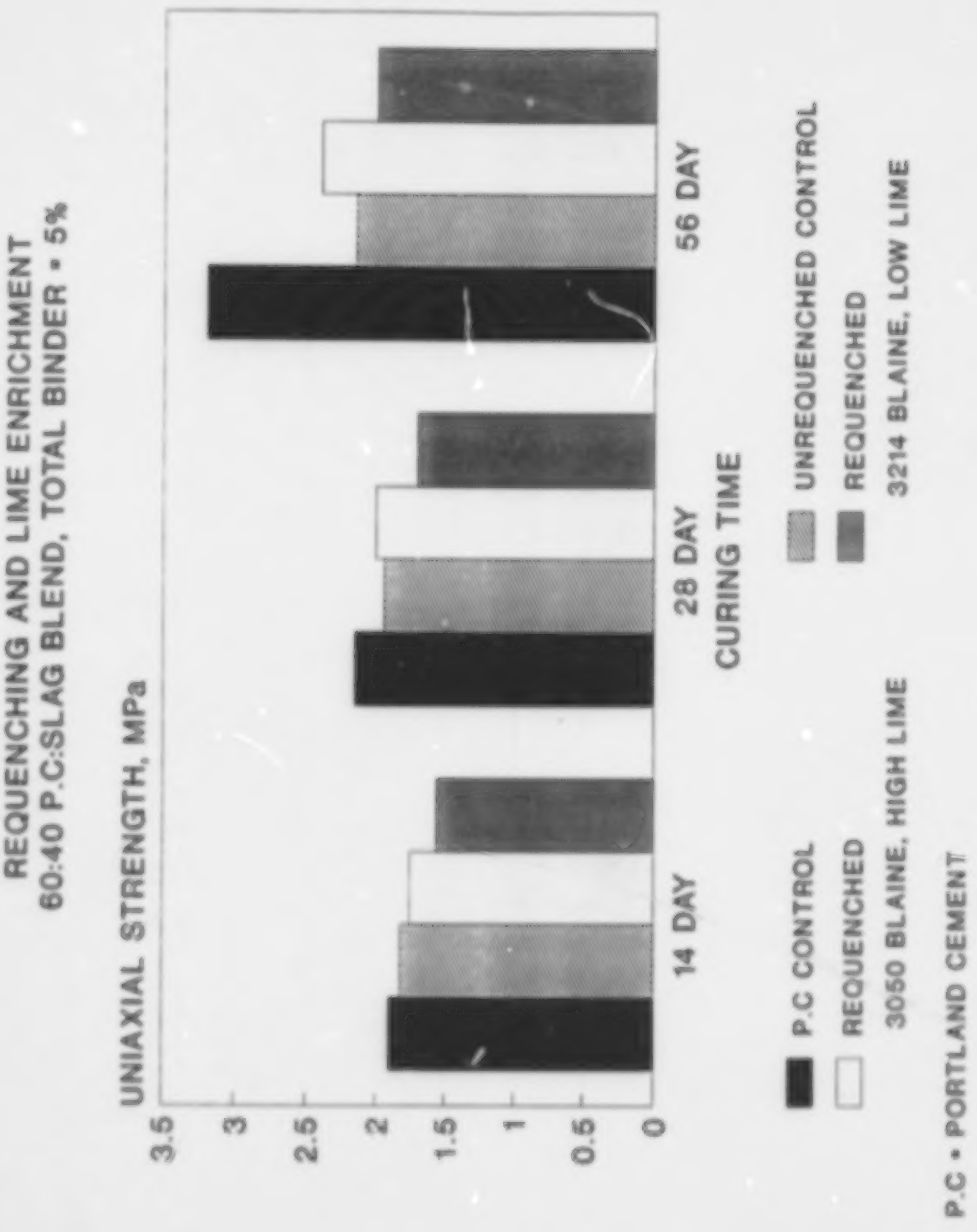


Figure 8.18, Effect of requenching and lime enrichment on copper slag performance as a rockfill binder.

8.6.1.4 Varying the proportion of slag replacement

A series of tests was conducted to confirm the results of the initial work on the reactivity of slag in the presence of portland cement, to investigate the rate of increase in compressive strength with varying amounts of copper slag but constant portland cement content (report 8, appendix A). These tests were conducted with total binder contents close to that which would be used in practical rockfilling, 5% to 11%. A constant portland cement addition of 3% of aggregate by weight was maintained and increasing amounts of ground slag added. If the required rockfill strength is achieved with portland cement at a 5% addition rate, the test aimed to determine how much additional ground slag was needed to achieve the same strength when only 3% portland cement was used. If ground copper slag can be produced at a lower cost than the purchase price of portland cement, the economics of saving 2% portland cement addition by adding slag might be favorable, depending upon just how much additional slag is required.

Test samples were 3 inch diameter cylinders. The copper slag used was not lime enriched and was ground to 3000 Blaine, previous work having indicated that these factors did not significantly affect the performance of the slag as binder.

The results of the test work are presented in table 8.18 and figure 8.19. They indicate that the copper slag had a moderate cementitious property if the replacement of the portland cement was below 40%, and that strength could be improved by increasing the slag to portland cement ratio. The amount of water in the mix played a very important role in developing cementation, as one would expect from concrete technology. Further work should concentrate on determining the optimum water content.

1. By adding 6% slag and limiting the amount of water used a strength equal to the control at 28 days and exceeding control at 56 days was obtained (20% better).

2. With 2% slag addition (40% replacement of portland) the strength was lower than the control. The difference was less at 56 days curing than at 28 days, i.e 35% at 28 days and 24% at 56 days.

3. Adding 4% slag increased the strength but it was still lower than the control. Doubling the slag addition produced only a 33% increase in strength at 28 days and only 15% at 56 days curing.

4. Adding 6% and 8% slag without adjusting the water content produced a 'soupy' mix with

approximately 20% excess water and no improvement in the strength from the additional slag.

5. Reducing the water content at high slag addition rates resulted in high strengths. At 8% slag addition there was still excess water present so strengths were disappointing.

Binder Composition binder % by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	 56 day
5% portland cement, (control)	2.67	3.02
3% portland & 2% slag	1.80	2.30
3% portland & 4% slag	2.40	2.65
3% portland & 6% slag	2.20	2.40
3% portland & 8% slag	1.20	1.73
3% portland & 6% slag, 20% less water	2.65	3.63
3% portland & 8% slag, 20% less water	1.70	2.04

Table 8.18, Compressive strength of rockfill with increasing slag addition.

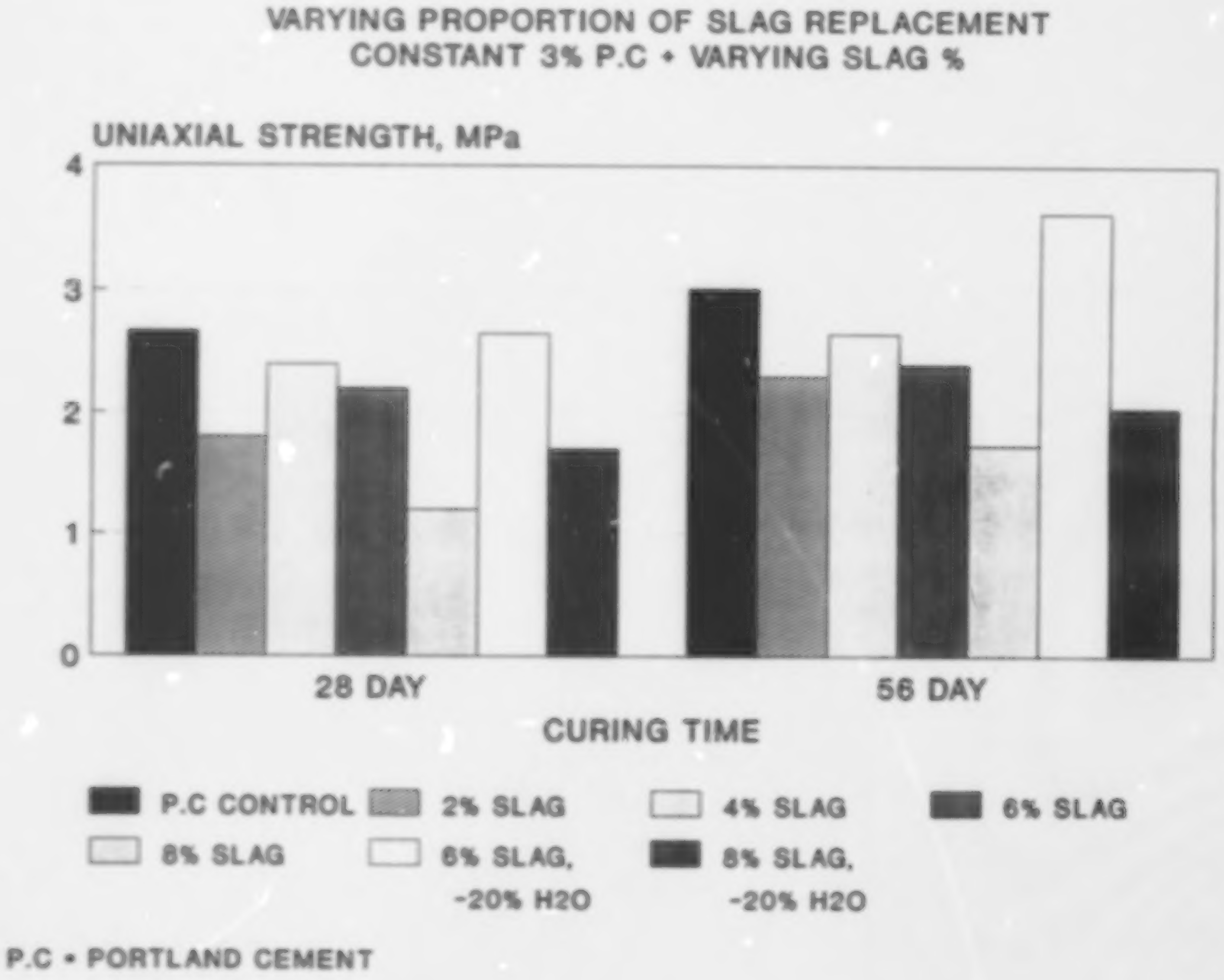


Figure 8.19, Performance of copper slag as a binder at various addition rates.

8..5 Effect of acidulation on slag performance

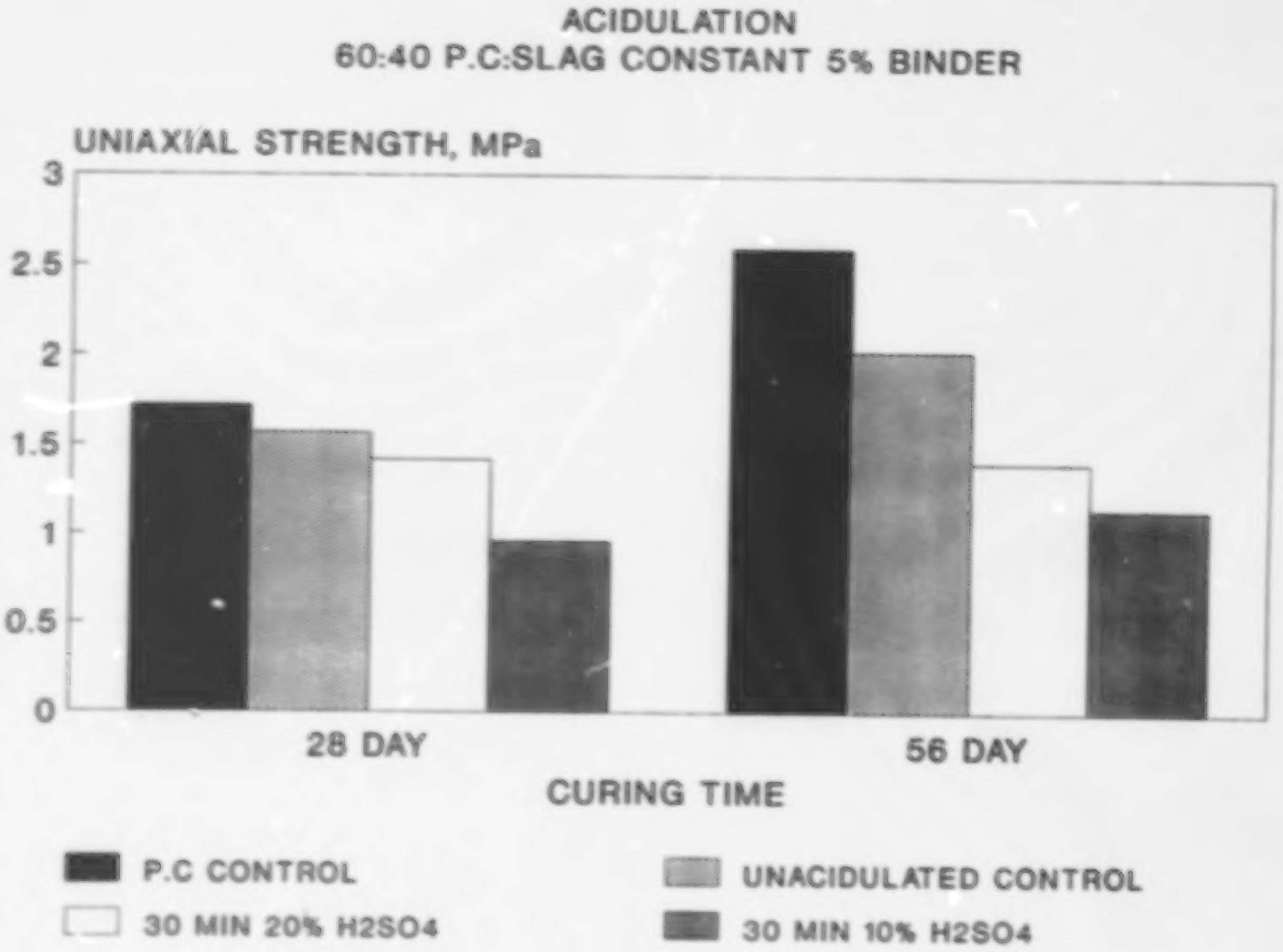
The results of the previous tests indicate that the ground copper slag is moderately reactive. A number of possibilities for improving the reactivity of slag exist which may change the economics of its use. The acidulation of the slag was one of these posibilities tested (report 7, appendix A).

Ground slag at 3000 Blaine size was washed in H_2SO_4 acid for 30 minutes and then used to replace 40% portland cement in a binder mix. 3 inch cylinders were cast and the compressive strength measured.

The results were disappointing, the acidulated slag returning lower strength than non acidulated slag.

Binder Composition (5% binder by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	56 day
5% portland cement, (control)	1.72	2.60
60% portland & 40% un acidulated slag (control)	1.57	2.03
60:40 portland:slag acidulated 30 min in 20% H_2SO_4	1.42	1.41
60:40 portland:slag acidulated 30 min in 10% H_2SO_4	0.96	1.15

Table 8.19. Compressive strength of rockfill with acidulated slag as a binder.



P.C • PORTLAND CEMENT

Figure 8.20, Effect of acidulation on copper slag performance as a rockfill binder.

8.6.1.6 Addition of gypsum, lime and anhydrite

Another possibility for improving the reactivity of ground copper slag is to add ground gypsum, lime or anhydrite. The potential of enriching the lime content by quenching in a lime solution has already been described. The addition of lime as a physically separate component in the binder is successful in improving copper slag reactivity at other backfilling operations (ie Mount Isa Mines) and hence it was expected to make a positive contribution to the performance of Kidd Creek copper slag.

The tests (report 9,10, appendix A) showed that the addition of a mixture of gypsum and lime to the slag resulted in a significant strength improvement. There is a limit to the improvement, adding gypsum and lime beyond a certain limit did not result in any further strength increase. The rate of strength development increased when gypsum and lime were added, ie higher early strength. This has some important practical implications for operations which can not allow a long curing time for their backfill.

The performance of anhydrite as an additive to improve slag performance was equally successful. Not only was final strength considerably higher but the rate of strength improvement was also increased.

The additives were tested on a binder mix of 2.5% portland cement with 5% ground slag (% of aggregate weight). This proportion of 2:1 slag to portland cement having been shown previously to give good performance and the 7.5% total addition rate would be competitive in cost with portland cement added at 5%. With the use of gypsum, lime and anhydrite additives this mix could approach and even surpass the strength achieved with 5% portland cement binder.

Two separate tests were conducted. The compressive strength of 3 inch cylinders was measured.

For the first test;

1. The addition of 1% gypsum and 0.65% lime to slag resulted in a 62% improvement in strength (for the same portland and slag contents), but adding 0.5% extra gypsum and 0.35% lime had minor effects on strength. A maximum effective addition rate is indicated.

2. The rate of strength development is much greater when gypsum and lime is added to the slag (a 70% increase). The control cylinders showed a greater strength improvement from 28 to 56 days curing time than did the cylinders with gypsum and lime added which attained most

strength in the first 28 days. The addition of gypsum and lime would be beneficial in increasing the early strength of fill, allowing a faster mining cycle.

Binder Composition (binder % by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	 56 day
5% portland cement, (control)	3.05	3.62
2.5% portland & 5% slag (control)	1.90	1.82
Add 1% gypsum & 0.65% lime	2.58	2.95
Add 1.5% gypsum & 1% lime	2.47	3.10

Table 8.20, Test 1. Compressive strength of rockfill with gypsum and lime added.

Note: Addition was to 2.5% portland and 5% slag blended binder.

For the second test;

1. The addition of anhydrite, gypsum or lime increased the strength of the test cylinders compared to that achieved with a straight slag and portland blend.

2. Some spectacular improvements in strength were observed with the addition of anhydrite. When 3% anhydrite was added the strength improved 217% and 240% at 28 and 56 days respectively over the slag and portland blend control. When just 1% anhydrite and 0.65% lime were added the strength improvement was nearly the same.

3. By adding anhydrite the slag and portland blends were able to match or better the strength attained by straight portland cement control.

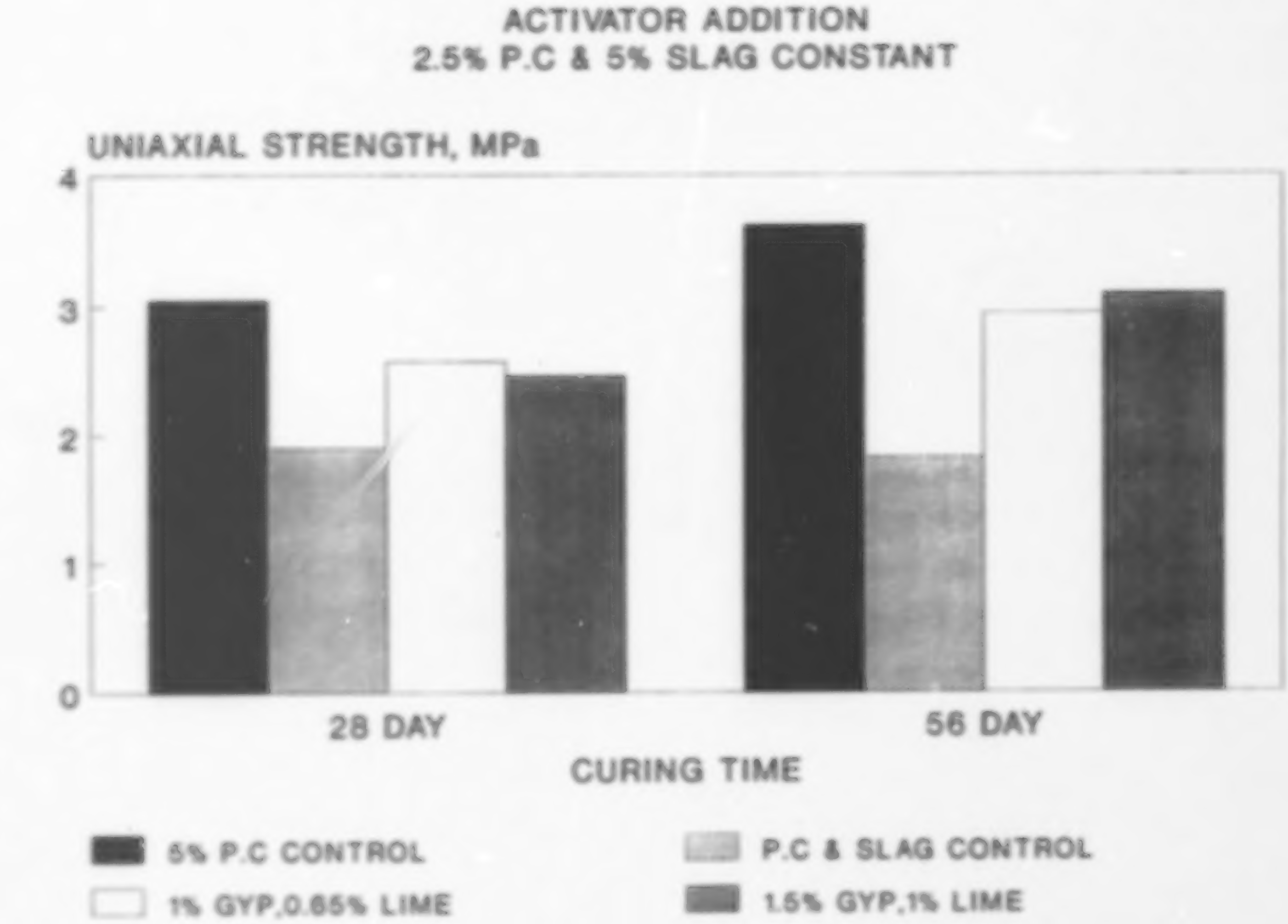
Binder Composition	Strength, MPa, Achieved at Various Curing Times	
(binder % by weight of aggregate)	28 day	56 day
5% portland cement, (control)	3.15	3.25
2.5% portland & 5% slag (control)	1.45	1.75
Add 1% gypsum & 0.65% lime	2.35	2.95
Add 1% gypsum	2.31	2.90
Add 3% gypsum	2.44	2.60
Add 1% anhydrite & 0.65% lime	3.13	4.00
Add 1% anhydrite	2.70	3.23
Add 3% anhydrite	3.15	4.15

Table 8.21. Test 2. Compressive strength of rockfill with gypsum, lime, anhydrite added.

Note: Addition was to 2.5% portland and 5% slag blended binder.

Figures 8.21 and 8.22 illustrate the above results.

The blends tested, especially the 3% anhydrite blend, shows good potential for being an economic binder competitive with portland cement. The chemistry of the blends has not been investigated in depth. Though these results are specific to Kidd Creek Mines copper slag the implications for other slags are significant. Lime is commonly trialed as an additive for slags, but gypsum and anhydrite less often. These results indicate that perhaps the potential of gypsum and anhydrite should be investigated more frequently.



P.C = PORTLAND CEMENT, GYP = GYPSUM

Figure 8.21, Effect of gypsum and lime on copper slag performance as a rockfill binder, test #1.

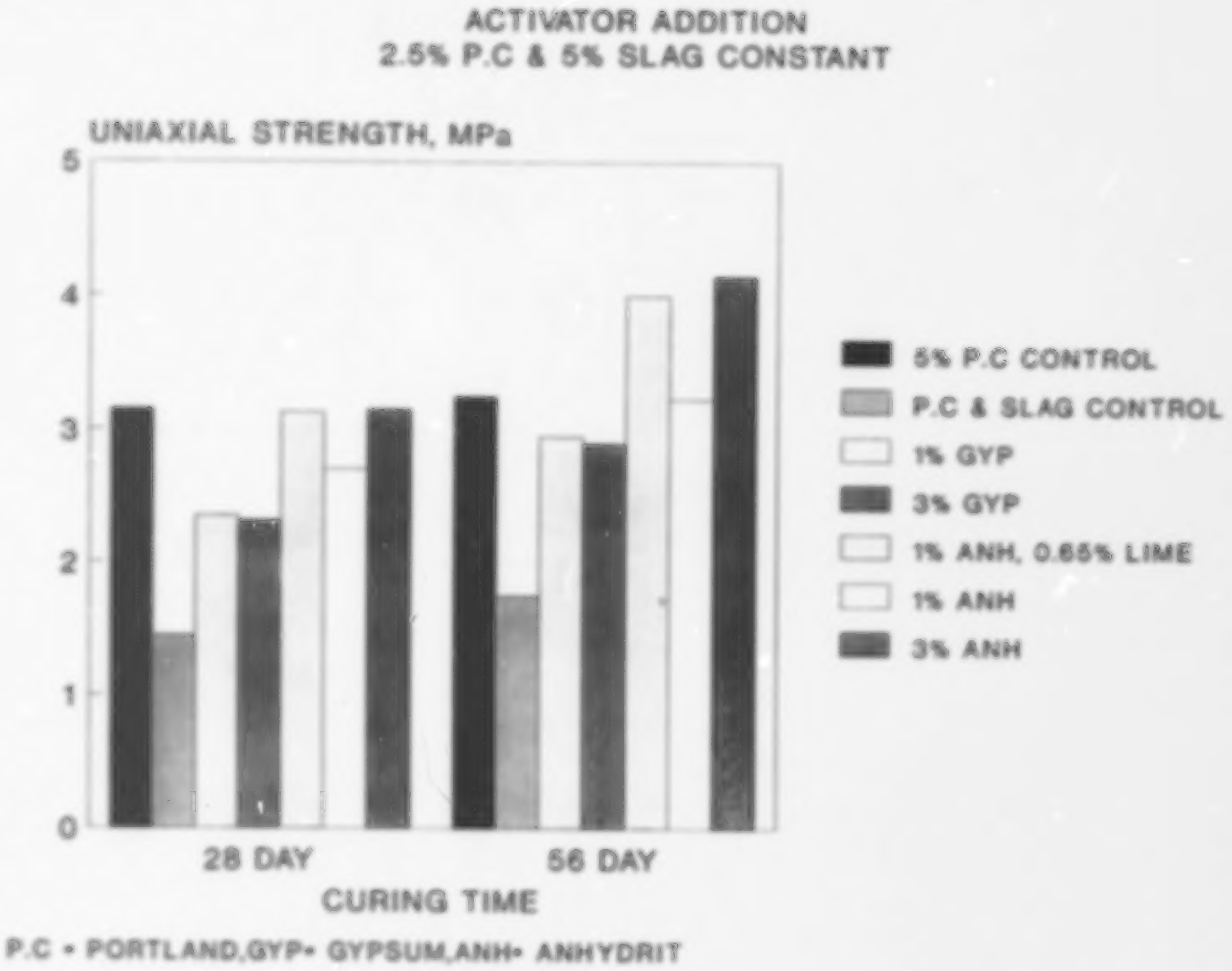


Figure 8.22, Effect of gypsum, lime and anhydrite on copper slag performance as a rockfill binder, test #2.

8.6.1.7 Effect of chemical accelerators

A variety of chemical accelerators commonly used in the concrete industry were trialed with ground copper slag and portland cement blends to determine if they showed any ability to speed the rate of strength development in rockfill (report 3,4,5, appendix A). Use of copper slag as a binder slows the rate of strength development considerably compared to portland cement. This is a major disadvantage when curing time for backfill is limited, it was hoped that the use of accelerators might overcome this.

The following chemical additives were trialed;

1. $FeSO_4$
2. K_2SO_4
3. $CaCl_2$
4. Daracell, a product of W.R. Grace Company
5. Pozzolith 133-HE, product of Master Builder Chemical Company

A series of three tests were conducted with these additives to determine their affect on the strength development of Kidd Creek Mines rockfill. The general conclusions from this test work were that the accelerators Daracell and Pozzolith worked, but the addition rates required and the cost of these products would make their use uneconomic. A less expensive product, $CaCl_2$, performed less well but could be cost effective in speeding the mining cycle. Some of the accelerators had optimum addition rates after which extra additions appeared to be detrimental to strength development. Figures 8.23 to 8.25 show the results of these tests.

Daracell gave the best performance, close to a 200% improvement in slag blend binder performance. It showed an optimum addition rate of 10%. See figure 8.25. The performance of Pozzolith 133-H, which was only used in the third test, also showed promise. These are high cost additives and unlikely to be cost effective.

$CaCl_2$ gave a 20% improvement in early strength but no long term increase. It increased the curing rate in the first month and could be used to get a shorter mining cycle. $CaCl_2$ is a reasonably inexpensive product, $380 per tonne, and it shows potential as a cost effective accelerator.

K_2SO_4 performed reasonably well in the first and second tests increasing the ultimate strength. The results from the third test were disturbing because they showed K_2SO_4 actually decreasing the strength.

$FeSO_4$ was disappointing, actually resulting in decreased strengths in many cases.

Test 1, (report 3, appendix A) measured the compressive strength of test cylinders after 6 days curing.

1. K_2SO_4 showed promise as an accelerator improving the 6 day strength compared to the slag and portland binder control, but the addition rate was important. At 10% addition the strength improved 265% but at 20% addition the improvement was only 197% indicating some optimum addition rate exists. The performance at 5% and 15% addition rates is investigated in test 2.

2. $FeSO_4$ did not perform well.

3. A point load tester had to be used to determine the low early strengths.

Binder Composition	Strength Achieved, MPa (6 day curing)
5% portland cement (control)	1.16
3% portland & 2% slag (control)	0.31
Add 10% $FeSO_4$	0.23
Add 20% $FeSO_4$	0.21
Add 10% K_2SO_4	0.82
Add 20% K_2SO_4	0.61

Table 8.22, Test 1, Compressive strength of rockfill with accelerators added.

Note: Accelerators were added to 3% portland and 2% slag blended binders.

Test 2 (report 4, appendix A) measured compressive strength of test cylinders at 28 and 56 days and included the accelerators Daracell and $CaCl_2$.

1. Addition of 2% slag to 3% portland improved the strength of cylinders 18% compared to that achieved with only 3% portland, but reached only 54% of the strength attained with 5% portland cement. This is consistent with previous results.

2. Daracell gave the best results of the accelerators tested, in one case exceeding the strength of the 5% portland cement control by 4%, (10% Daracell addition). The optimum addition rate for Daracell was determined to be around 10%, addition rates above and below this returning lower strengths.

3. K_2SO_4 performed well, as expected from the first test. Much of the improvement due to this accelerator occurred between 28 and 56 days of curing. An optimum addition rate was not apparent from this test.

4. $CaCl_2$ increased the strength of test cylinders by 16% at a 5% addition rate and 19% at a 3% addition rate. This indicates an optimum addition rate exists. The strength of cylinders with 5% $CaCl_2$ was 90% of the 5% portland control at 28 days but only 63% at 56 days. The accelerator works to improve early strength but longer term strength is not improved so much.

5. $FeSO_4$ did not contribute to the compressive strength of the mixes after 56 days of curing, it even decreased the strength when a 5% addition rate was used.

Binder Composition	Strength, MPa, Achieved at Various Curing Times	
(binder % by weight of aggregate)	28 day	56 day
5% portland cement, (control)	2.37	3.18
3% portland (control)	1.14	1.46
3% portland & 2% slag (control)	1.73	1.73
Add 5% K_2SO_4	1.75	2.42
Add 10% K_2SO_4	1.50	2.05
Add 15% K_2SO_4	1.87	2.34
Add 5% Daracell	1.68	2.18
Add 10% Daracell	2.81	3.30
Add 15% Daracell	2.24	2.90
Add 5% $FeSO_4$	1.22	1.42
Add 10% $FeSO_4$	1.77	1.70
Add 3% $CaCl_2$	2.04	2.04
Add 5% $CaCl_2$	2.14	2.00

Table 8.23, Test 2. Compressive strength of rockfill with accelerators.

Note: Accelerators added to 3% portland and 2% slag binder blend.

Test 3, (report 5, appendix A), again considered 28 and 56 day cured compressive strengths. It included an assessment of Pozzolith 133-HE as an accelerator.

1. Pozzolith 133-HE was an effective accelerator, improving the strength of a slag and portland blend by 51% after 56 days of curing.

2. The results of this test in combination with the previous one indicate that the optimum percentage of Duracell in the mix is around 10%. There was a 40% decrease in strength when

the Daracell concentration was increased from 7.5% to 12.5%.

3. The results of this test in combination with the previous tests indicate that the optimum $CaCl_2$ concentration in the mix is around 3%.

4. Results of K_2SO_4 use were disappointing, decreasing strength considerably after 28 and 56 days curing. This is in contradiction to the previous tests.

Binder Composition (binder % by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	56 day
5% portland cement, (control)	4.60	6.20
3% portland & 2% slag (control)	2.50	3.50
Add 7.5% Daracell	2.83	4.50
Add 12.5% Daracell	1.86	2.70
Add 5% Pozzolith 133-HE	2.25	3.70
Add 10% Pozzolith 133-HE	3.90	5.30
Add 2% $CaCl_2$	3.01	3.40
Add 7% $CaCl_2$	2.71	2.90
Add 2% K_2SO_4	1.98	2.06
Add 4% K_2SO_4	2.56	2.81

Table 8.24, Test 3. Compressive strength of rockfill with accelerators.

Note: Accelerators added to 3% portland and 2% slag binder blend.

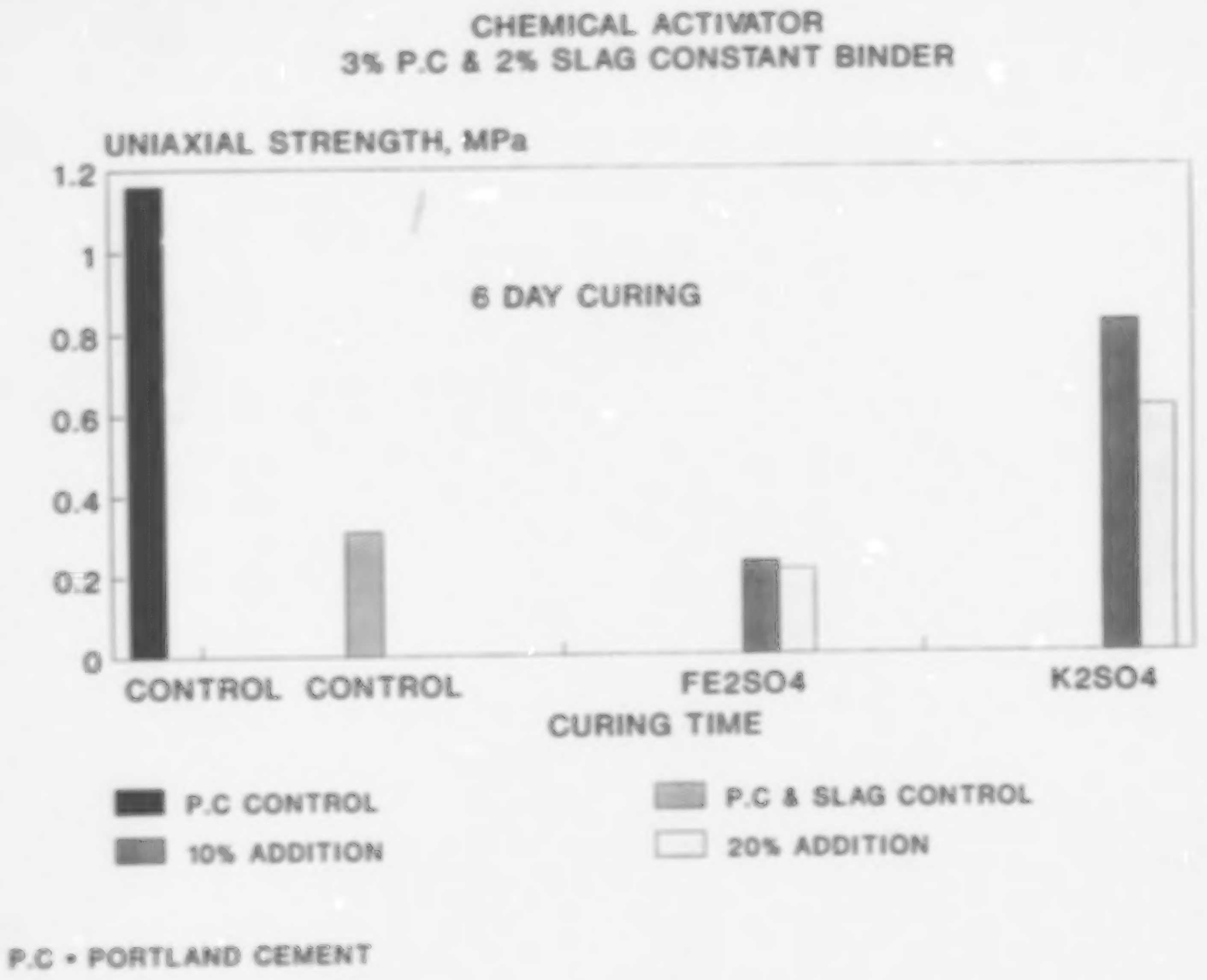


Figure 8.23, Effect of chemical accelerators on copper slag performance at an early age.

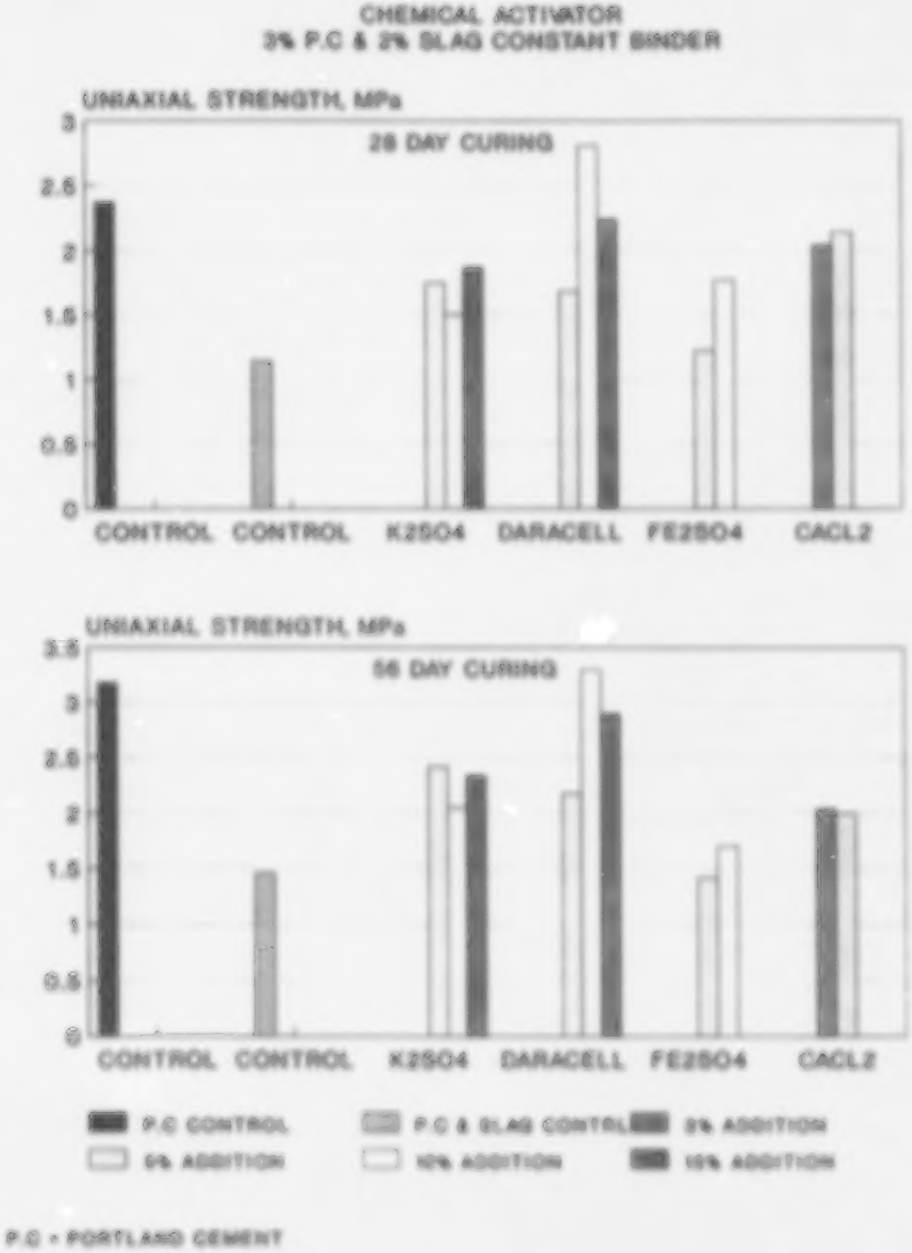
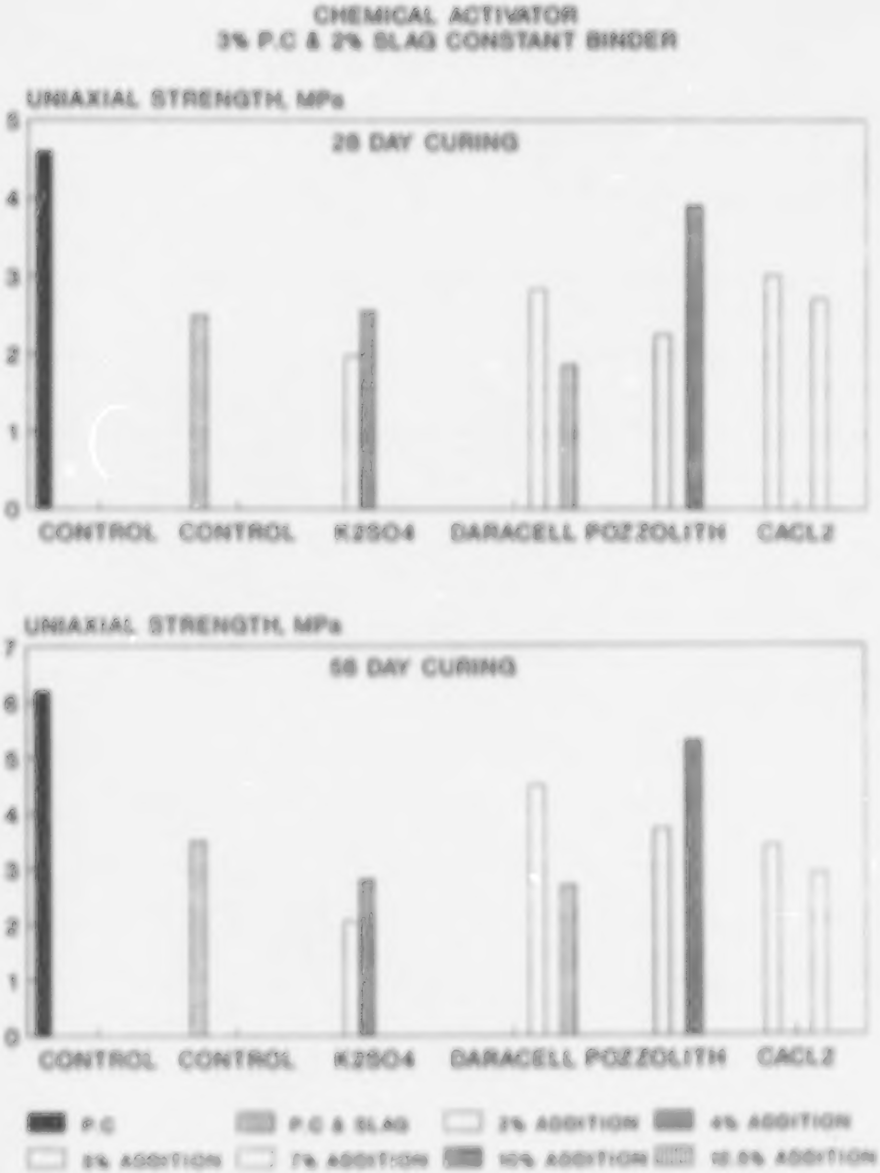


Figure 8.24, Effect of chemical accelerators on copper slag performance as a rockfill binder.



P.C = PORTLAND CEMENT

Figure 8.25, Effect of chemical accelerators on copper slag performance as a rockfill binder.

8.6.1.8 Grindability of Kidd Creek copper slag

The economics of ground copper slag use as a portland cement replacement are principally dependent upon the cost of grinding it to a suitable size. The test work described above has indicated that a suitable binder can be made by blending ground slag and portland cement, that the addition of anhydrite or other chemicals can improve the performance of the blend significantly and that the slag should be ground to about 3000 Blaine size. Joint research by Kidd Creek Mines and CANMET, partly under the auspices of this project , has indicated that the copper slag is relatively difficult to grind.

8.6.2 Test work on Flyash as a Rockfill Binder

It is known from concrete technology that the addition of 10% to 15% flyash by weight of portland cement will produce a high strength concrete and that the long term strength development will be improved. Kidd Creek Mines have conducted much research over the years into the potential for flyash to replace a portion of the portland cement as rockfill binder. The principal problem with flyash is that at high replacement rates the curing times for the rockfill are adversely affected and the rockfill is slow to develop strength. The same problem was identified with the use of ground copper and blast furnace slags as a backfill binder.

Much of the test work reported here was conducted in conjunction with the work on copper slag. In general the flyash blend binders performed better than the copper slag blend binders.

8.6.2.1 Effect of acidulation on flyash performance

Flyash normally has a high iron and or aluminum content which is undesirable in a cementitious material. The concept of flyash acidulation is that the oxides of excess metals can be turned into soluble salts which can then be washed away. Removal of the undesirable metals should create a much greater surface area for the reaction of silica and hence a stronger cement. Hydrochloric and sulphuric acids were used for these tests. Type F flyash was used for these tests. Type F flyash had shown inferior performance compared to type C in previous tests so there was an incentive to attempt an improvement.

The results (report 6, appendix A) indicated that acidulation could have a slight positive affect on the performance of flyash as a binder, but that the strength and type of acid, and time of acidulation are important factors. Hydrochloric acid reduced the strength of test cylinders and the use of high strength acid completely killed the cementitious reactivity of the binder so that no strength was ever developed.

Acid dosage was by weight; 1.2 times the weight of flyash. The acid was allowed to chemically act on the flyash for 15 or 60 minutes after which excess acid was decanted. A calculation of the amount of acid retained in the flyash was made and an appropriate adjustment made in the volume of water added. Even with this adjustment the net moisture content of the acidulated samples was about 1% higher than the unacidulated ones which most likely caused the decrease in acidulated sample strength.

1. 66% replacement of the portland cement with type F flyash in the binder reduced the sample strengths by 56%, as was expected.

2. Acidulation of flyash with H_2SO_4 at 20% concentration for 60 minutes resulted in a 70% increase in strength over that obtained with the unacidulated control.

3. Acidulation in high concentrations of acid caused the strength to be reduced. Acidulation with 38% concentration resulted in no strength development at all.

4. HCl even at lower concentrations caused a reduction in strength, 53% less strength when 20% HCl for 60 minutes acidulation compared to the unacidulated control.

Binder Composition	Strength, MPa, Achieved at Various Curing Times	
	28 day	56 day
100% portland cement, (control)	1.86	2.46
33% portland & 66% type F flyash (control)	1.05	1.42
38% HCl, 60 min.	0.00	0.00
38% HCl, 15 min.	0.00	0.00
20% HCl, 60 min.	0.39	0.67
20% HCl, 15 min.	0.85	1.40
38% H_2SO_4, 60 min.	0.00	0.00
38% H_2SO_4, 15 min	0.00	0.00
20% H_2SO_4, 60 min	1.76	2.40
20% H_2SO_4, 15 min	1.80	1.57

Table 8.25. Compressive strength of rockfill with acidulated type F flyash.

Note: Acidulation is of 33% portland and 66% type F flyash blended binder.

A further test (report 7, appendix A) of the affect of acidulation on flyash was conducted to confirm the positive results obtained with 20% H_2SO_4.

1. Replacement of 66% of portland by type F flyash in the binder reduced the strength by 60%, which is consistent with results obtained in the first test.

2. The effect of acidulation in H_2SO_4 at 10 and 20% concentrations did not result in the same strength increases obtained in the initial test. A small increase was noted but not significant.

Binder Composition	Strength, MPa, Achieved at Various Curing Times 28 day	56 day
100% portland cement, (control)	1.72	2.60
33% portland & 66% type F flyash (control)	1.03	1.66
20% H_2SO_4, 30 min	1.16	N/A
10% H_2SO_4, 30 min	1.10	1.60

Table 8.26, Compressive strength of rockfill with acidulated type F flyash.

Note: Acidulation is of 33% portland and 66% type F flyash blended binder.

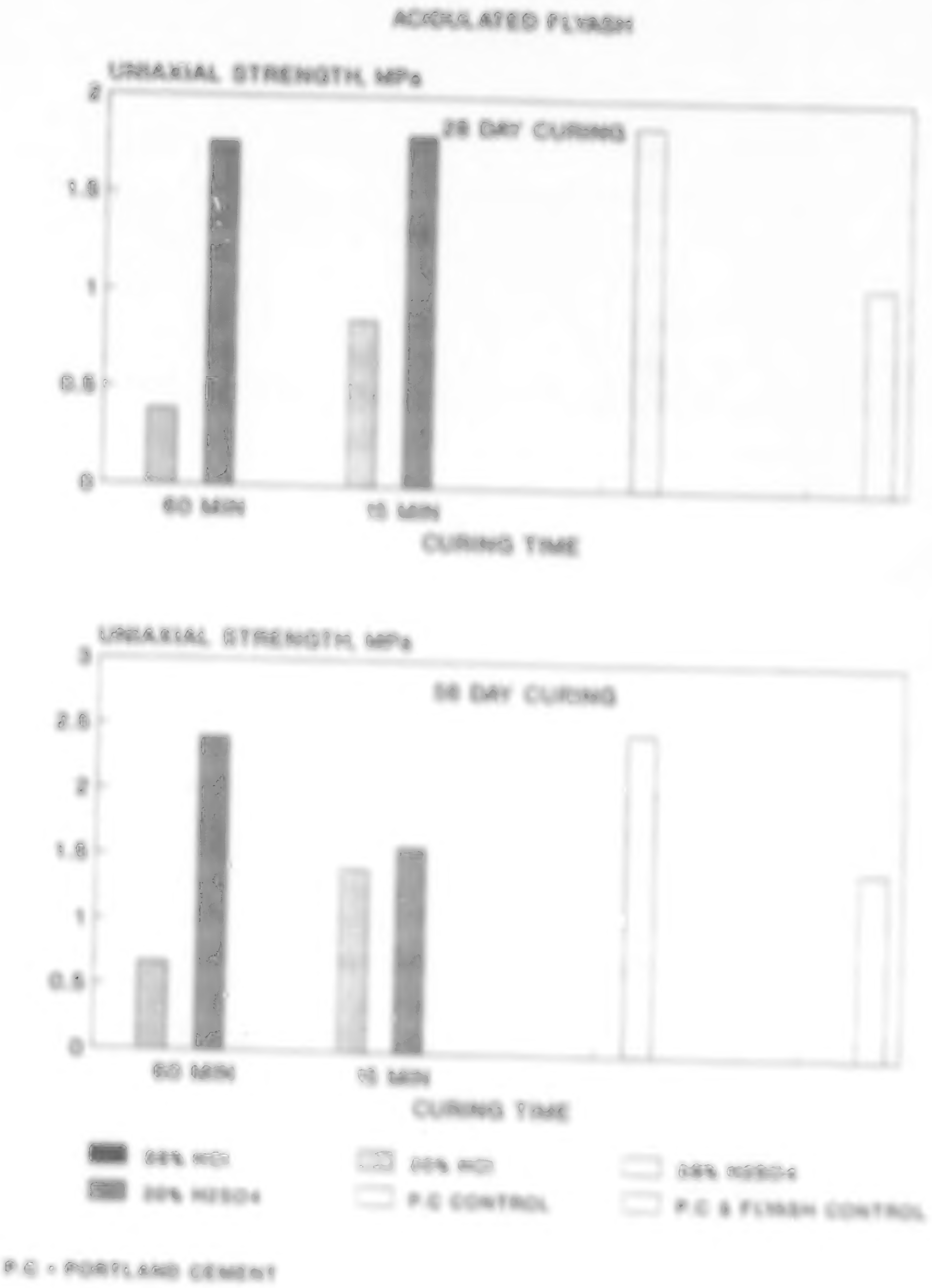


Figure 8.26, Effect of acidulation on performance of type F flyash as a binder, test #1.

8.6.2.2 Addition of gypsum, lime, anhydrite and sodium silicate

The potential to improve the performance of flyash as a binder by the addition of gypsum, lime, anhydrite and sodium silicate to the binder blend was investigated. These tests concerned type C flyash as the most promising type. The tests were conducted in conjunction with similar tests on copper slag as a binder. The performance of the flyash was much superior to that of copper slag.

The tests (report 9,10, appendix A) indicate that gypsum and anhydrite are capable of improving the strength of rockfill bound with a portland cement and flyash blend. Sodium silicate decreased the strength of rockfill when added to the binder. Lime was tested in combination with gypsum and anhydrite but it had no significant influence on the strength development. In general anhydrite and gypsum performed better on their own than in combination with lime. An optimum addition rate for gypsum, lime and anhydrite exists. Additions at a rate higher than the optimum appear to be detrimental to the strength of the rockfill.

Of particular interest is the ability of both gypsum and anhydrite to improve the early strength of the rockfill. Ultimate strength is increased, but most of the improvement occurs by 28 days curing.

The water cement ratio is upset by the addition of gypsum, lime and anhydrite to the blend. This is a possible cause of the loss in strength noticed at higher addition rates. Some more testing may be warranted to investigate this.

The test work has shown that gypsum and anhydrite addition can have a beneficial affect on the performance of flyash as a rockfill binder. Various Canadian suppliers of gypsum and anhydrite were queried on the cost of these materials. In 1989 Canadian dollars the approximate cost of gypsum was $90 per tonne and the cost of anhydrite was $110 per tonne. These prices are F.O.B at Timmins Ontario.

The first test (report 9, appendix A) investigated the potential of gypsum, lime and sodium silicate.

1. Gypsum and lime in combination gave a moderate increase in strength over the control. Increasing the addition rate gave a proportional improvement in the strength.

2. The performance of sodium silicate was disappointing. It caused a reduction in strength. However the rapid strength development shown indicates that it might be applicable as a way of achieving high early (2 to 7 days) strength.

Binder Composition (binder % by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	 56 day
5% portland cement, (control)	3.05	3.62
2.5% portland & 2.5% type C flyash (control)	5.20	6.02
Add 0.65% gypsum & 0.33% lime	6.05	6.23
Add 1% gypsum & 0.5% lime	7.00	7.10
Add 5% sodium silicate	4.45	4.12
Add 10% sodium silicate	5.33	4.20

Table 6.27, Test 1, compressive strength of rockfill with type C flyash, gypsum, lime and sodium silicate.

Note: Addition was to 2.5% portland and 2.5% type C flyash blended binder.

The second test (report 10, appendix A) investigated gypsum, lime and anhydrite.

1. By adding gypsum, lime and or anhydrite it was possible for a blended binder of flyash and portland to exceed the strength of the straight portland control strength. The highest strength obtained was with 1% anhydrite and 0.65% lime addition, 52% stronger than the straight portland control at 56 days of curing.

2. The rate of strength development was generally superior when the gypsum, lime and anhydrite were added, by 20 to 30% from 28 to 56 days.

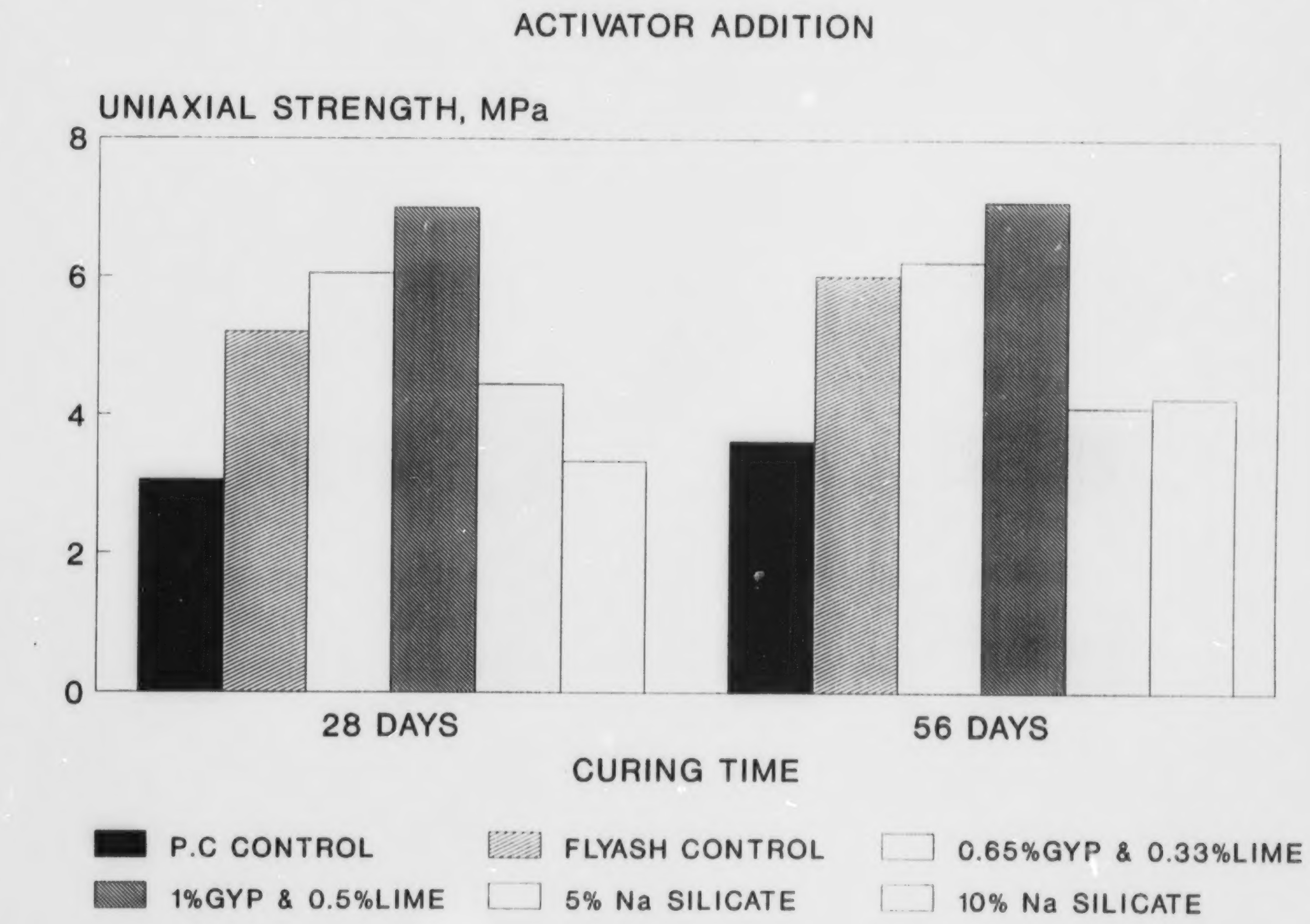
3. Gypsum addition was best when added at 1% with no extra lime. Extra lime reduced the effectiveness of gypsum at 56 days curing (12% less strength) although it did cause an increase in 28 day strength (18%). 3% gypsum addition was not as effective as 1% addition, a 58% reduction in strength, which could have been due to insufficient water being present for complete reaction to take place.

4. 1% anhydrite addition gave the best results. The addition of lime made no major impact on the performance of anhydrite. Anhydrite added in excess of 1% had a detrimental affect on strength compared to the 1% results.

Binder Composition (binder % by weight of aggregate)	Strength, MPa, Achieved at Various Curing Times 28 day	56 day
5% portland cement, (control)	3.15	3.25
2.5% portland & 2.5% type C flyash (control)	2.78	2.92
Add 1% gypsum & 0.65% lime	2.96	3.95
Add 1% gypsum	3.5	4.46
Add 3% gypsum	1.78	1.90
Add 1% anhydrite & 0.65% lime	4.02	4.93
Add 1% anhydrite	4.90	4.80
Add 3% anhydrite	2.13	2.55
Add 5% anhydrite	2.20	3.15

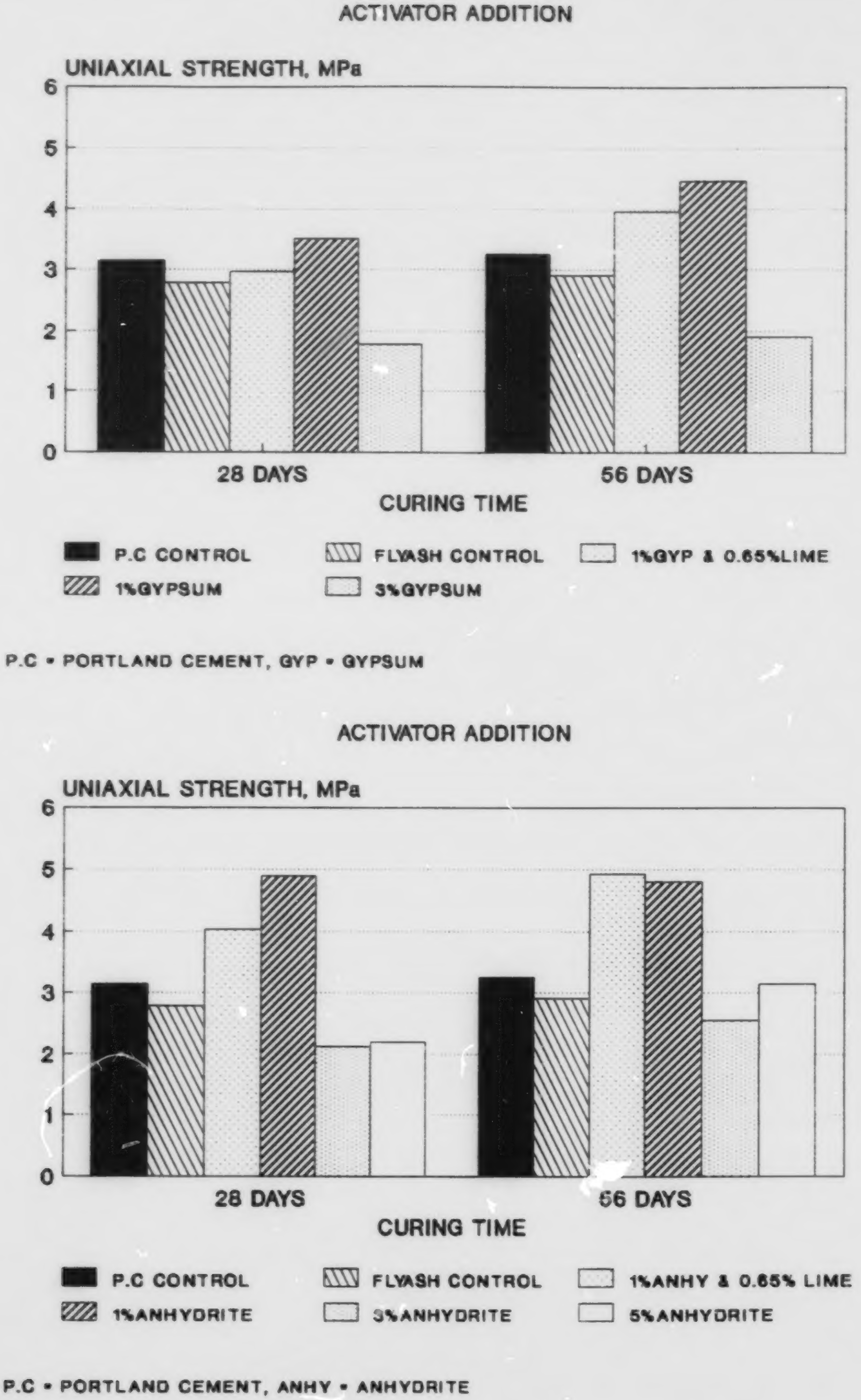
Table 8.28, Test 2, compressive strength of rockfill with type C flyash, gypsum, lime and anhydrite.

Note: Addition was to 2.5% portland and 2.5% type C flyash blended binder.



P.C = PORTLAND CEMENT, GYP = GYPSUM

Figure 8.27, Effect of gypsum, lime and sodium silicate on type C flyash performance as a rockfill binder, test#1.



P.C = PORTLAND CEMENT, GYP = GYPSUM

P.C = PORTLAND CEMENT, ANHY = ANHYDRITE

Figure 8.28, Effect of gypsum, lime and anhydrite on type C flyash performance as a rockfill binder, test #2.

8.7 ANHYDRITE AS A HYDRAULIC FILL BINDER ALTERNATIVE

Anhydrite($CaSO_4$) will, in the presence of water, form dihydrite ($CaSO_4.2H_2O$) a strong material. The transformation is relatively slow.

In order to investigate the potential of anhydrite as a binder alternative for hydraulic tailings fill a series of laboratory tests were commissioned as a sub contract to this project. The work was carried out by the Department of Mining and Metallurgical Engineering, McGill University, Montreal, Quebec. The full report is available in appendix H, only a shor[illegible]ew of the results are presented here. The test work was conducted in unison with a review of monolithic packing materials. The conclusions from this review of monolithic packing are discussed in Chapter 6.

8.7.1 Test Work

Anhydrite, tailings, accelerator and water were mixed in varying proportions. The compressive strength, elasticity and density of a cylinder cast from each mixture was measured. The accelerator was ferro sulphate ($FeSO_4.7H_2O$). The reaction of anhydrite with water is too slow for practical mining purposes without the addition of an accelerator. Table 8.29 below shows the various mixtures which were tested.

Variable				
Anhydrite particle size range (mesh)	-20 to +50	-50 to +100	-100	-200
Anhydrite:Tailing	25:75	50:50	75:25	100:0
Accelerator (parts per total parts)	2.8	3.6		
Water at 19 - 20° C (parts per total parts)	25	30	35	

Table 8.29, Variables in anhydrite tailings mixtures

All combinations of the above range of variables were tested.

8.7.2 Results

The results of unconfined testing on all the combinations is presented in table 8.15. The following general conclusions can be made.

1. As the mesh size of the anhydrite decreased so the unconfined compressive strength increased. The maximum strength values were obtained when using minus 200 mesh size anhydrite.

2. An optimum accelerator addition rate of 2.8 parts by total weight of the mix was determined. Additions beyond this did not improve the strength of the mix.

3. Decreasing the anhydrite to tailing ratio decreased the strength. This was especially pronounced after 28 days of curing.

4. The unconfined compressive strength was directly proportional to the curing time up to 28 days. More test work is required to determine how linear is the hardening rate after this time.

5. Pumpability of the mix can be ensured with 25 parts water by weight. This holds even for the largest sizes of anhydrite tested, minus 20 to plus 50 mesh size.

6. Increasing the water content form 25 to 30 parts by weight did not change the strength appreciably. However, increasing the water content to 42 parts by weight (i.e 41% increase) resulted in a 45% decrease in the average strength.

The mixtures which showed most promise of being cost effective were;

1. 50 parts natural anhydrite sized from minus 50 to plus 100 mesh
   50 parts tailings
   2.8 parts accelerator
   25 parts water

This achieved a; 28 day strength of 8.9 MPa
modulus of elasticity, E, of 1.1 GPa
density of 1600 kg/m^3

2. 25 parts natural anhydrite sized to minus 100 mesh
   75 parts tailings
   2.8 parts accelerator
   25 parts water

   This achieved a; 28 day strength of 5.7 MPa
   modulus of elasticity, E, of 1.4 GPa
   density of 1894 kg/m^3
3. 50 parts natural anhydrite sized to minus 100 mesh
   50 parts tailings
   2.8 parts accelerator
   25 parts water

   This achieved a; 28 day strength of 15.9 MPa
   modulus of elasticity, E, of 1.9 GPa
   density of 1900 kg/m^3

Using minus 200 mesh anhydrite is not recommended due to excessive cost of milling the rock, even though it results in good compressive strength.

8.8 FALCONBRIDGE SUDBURY OPERATIONS EXPERIENCE WITH FERROUS SLAG AS A HYDRAULIC FILL BINDER

As a consequence of research work carried out for this project and reported earlier in this chapter Falconbridge Sudbury operations commenced using finely ground blast furnace slag as a binder for hydraulic fill (slag "A"). The introduction of this binder alternative which appeared so promising in laboratory testing has been characterized by a number of practical problems. These problems have encouraged the realization that what appears to be a relatively simple process of backfilling is in fact a rather complex inter-dependent system. Any change in a component of the system is bound to upset the equilibrium, and so changes must be recommended only when they have been analyzed for their total impact. The enthusiasm for a change based upon laboratory work must be tempered by a sober review of how it may affect the system as a whole. Dollars saved in one area are quickly lost in another dependent area if the change is considered in isolation.

8.8.1 Fineness of slag

The blast furnace slag used by Falconbridge is ground to a very fine size, around 4600 Blaine size. This compares to normal portland cement which is ground to around 3000 Blaine size. The fineness of grind of a cementitious product is a function of the reactivity required. The finer the grind, the greater the surface area and hence the higher the reactivity. The relationship is not proportional and each cementitious material will display its own particular characteristic in this respect. It just happens that the slag Falconbridge uses requires a fine grind.

The fineness of grind is important to slag cost. The more grinding energy required to reduce the slag particles to the appropriate size the more it costs to produce. So in general the slag manufacture will only grind as fine as is essential to make the product reactive.

Slag size becomes a problem in hydraulic filling for four reasons.

1. The fine slag particles will remain suspended in the transport water and it is difficult to retain them in the stope, so considerable quantities are lost with the drainage water.

2. The fine particle size of slag makes the segregation problem common to all hydraulic filling a magnitude worse. Segregation may cause a reduction in mass stability of the fill.

3. The fine particles of slag clog the drainage paths in the fill reducing permeability.

4. Segregation causes impermeable layers of slag to form reducing effective mass permeability.

It is difficult to control the loss of slag particles with the drainage water. The slag particles are so fine that the filter fabrics used on fill fences and drainage pipes are not efficient at retaining them in the stope. Fabrics with finer weave are difficult to justify because drainage rates will be adversely affected and hence filling rates would have to slow. The type of fabric used at Falconbridge has been adopted as a good compromise between drainage rate and filtering efficiency when using portland cement as a binder. Use of slag has upset this compromise. A finer fabric will tend to clog with the slag particles drastically reducing the drainage rate achieved.

The loss of slag with the drainage water is a loss of fill strength and of course money. Even though in laboratory testing the slag cement gave a superior strength fill compared to portland cement the potential exists for the effective strength in place underground to be significantly lower if the loss of slag with the drainage water is excessive. This is a major concern for the operations.

Segregation is a function of differential settling and momentum due to particle size differences in hydraulic fill. Because the size difference between binder and tailings is on average greater when using a slag cement than for portland cement the segregation problem is greater when using slag cement. More segregation implies less control of the fill mass and hence possible lower stability due plans of weakness.

All binders used for hydraulic fill will tend to reduce the permeability because of their small size which allows them to pack in and block drainage paths. Slag being a little finer packs a little better and hence reduces permeability a little more. If dewatering of a pour relies upon percolation drainage then this may become a problem as has been experienced in several pours at Falconbridge mines in Sudbury. Permeability is further reduced by the greater segregation encountered with the use of slag.

8.8.2 Curing rate of slag.

The slag used by Falconbridge mines has a characteristically slow curing rate. This is common to most cementitious slags.By itself this is not a major problem for the Falconbridge operations as the available curing time is reasonably long. However in combination with the problems associated with slag fineness some interesting disadvantages have become apparent.

Slag escaping from the stope with the drainage water will tend to settle in sumps and drain lines down stream in the mine. The slow curing characteristics of the slag mean that it is often still active at this stage and has caused the cementing up of sumps and drain lines at Falconbridge mines. This had not been noticed previously when using portland cements. Portland cement seems to be much less robust, when mixed with excess water it seems to hydrate quickly while still in suspension and so when it settles out it forms only a very weak cement if any thing at all. The smaller quantities of portland that escaped also meant the problem was not as noticeable. This ability of slag to harden in sumps and drain lines can become a major problem for mines, especially if the slag should reach main settlers and dewatering pump stations.

The higher level of particles in the drain water when using slag as a binder means that more wear can be expected on dewatering pumps.

The slower curing time inter acting with the reduction in drainage due to slag inhibiting percolation and due to increased segregation has caused problems. Previously with the use of portland cement as a binder lack of drainage problems were not noticed because the fast set of the cement. With slag poor drainage results in a soupy mess which is impossible to work on top of until the slow curing slag starts to develop strength. This has caused production delays, but more importantly such a poorly drained material with no cohesion is liable to liquefaction.

8.8.3 The solutions

Two methods are proposed for overcoming these problems.

1. Blending of a small amount (10%) of portland cement in with the slag.

2. Application of flocculants for high slag content pours.

The portland cement is intended to give the slag bound backfill some early cohesion so that liquefaction of undrained material is not a risk and to give just enough strength for workers to get back on top while the slag is still developing initial strength. The portland replaces the lime as an activator for the slag hydration process and results in the slag giving a little better strength, although this is not its primary role.

The flocculants are intended to reduce the loss of fine slag from the stope with the drainage water, reduce segregation by increasing the effective slag particle size and locking the slag to tailings particles, and improve the drainage performance of the fill. A discussion of the technology of flocculants and the methods for their addition is not relevant here. Operations using slag cements will often not consider placing fill unless flocculants are also being applied.

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2

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CANADA-ONTARIO
MINERAL DEVELOPMENT AGREEMENT

BACKFILL ALTERNATIVES IN ONTARIO MINES

VOLUME 2 - APPENDICES

Prepared by

FALCONBRIDGE LIMITED, Sudbury, Ont.

DSS FILE No. 09SQ.23440-6-9011

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APPENDIX A

REPORTS OF LABORATORY TEST WORK

CONDUCTED
BY

KIDD CREEK MINES LIMITED

592 PP

REPORT #1

BLAINE SIZE VS. COMPRESSIVE STRENGTH OF COPPER SLAG

OBJECTIVE: To study the effect of slag fineness size on the compressive strength development of the backfill material. A comparison between regular slag and requenched, lime-enriched, slag was also carried out.

PROCEDURE:

- Sample preparation followed the C39-83b ASTM standard.
- Each set was made of nine 3-in. diam. cylinders that were tested at 14, 28 and 56 day intervals. The materials used in these sets were :

SET A:

Aggregate = 10.26 kg
Sand = 5.08 kg
P.C. = 0.76 kg (100%)
Water = 0.63 kg

SET B:

Aggregate = 10.26 kg
Sand = 5.08 kg
P.C. = 0.51 kg (60%)
Slag = 0.3 kg (40%) , Cu regular slag, 3000 Blaine
Water = 0.63 kg

SET C :
Same as set B but 4000 Blaine slag size was used.

SET D :
Same as set B but requenched copper slag #1 of 3050 Blaine was used.

SET E :
Same as set B but requenched copper slag #2 of 3214 Blaine was used.

RESULTS : Following table presents the results obtained for each set at 14, 28 and 56 days. The results are also shown by graphs 1 and 2.

DAYS	A(MPa)	B	C	D	E
14	1.9	1.82	1.65	1.75	1.56
28	2.15	1.94	1.92	2.0	1.7
56	3.2	2.15	2.2	2.4	2.0

DISCUSSION:

- The tests obtained encouraging results indicating that the copper slag did contribute greatly to the strength development of slag/cement mixes. The Blaine size finer than 3000 had no effect on the strength development of the samples. Use of lime solution with requenched slag slightly increased the compressive strength of the material.

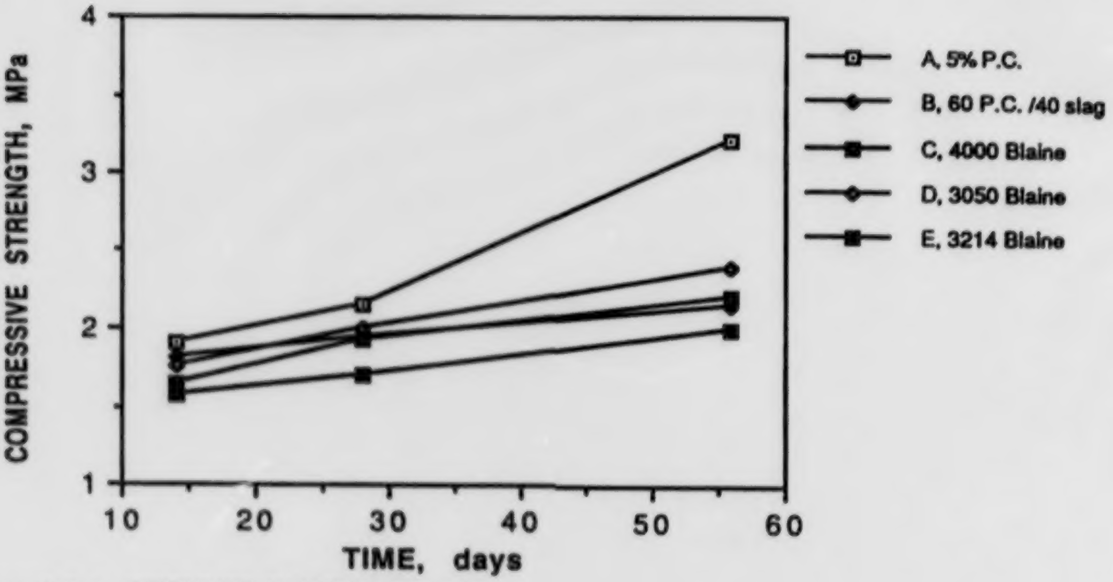
CONCLUSION :

- Sets B and D showed very encouraging results. The 14 and 28 day strengths of set B were 95 and 90% of the strength of the control set A respectively. However, the longterm strength of set B was only 68% of the control set.

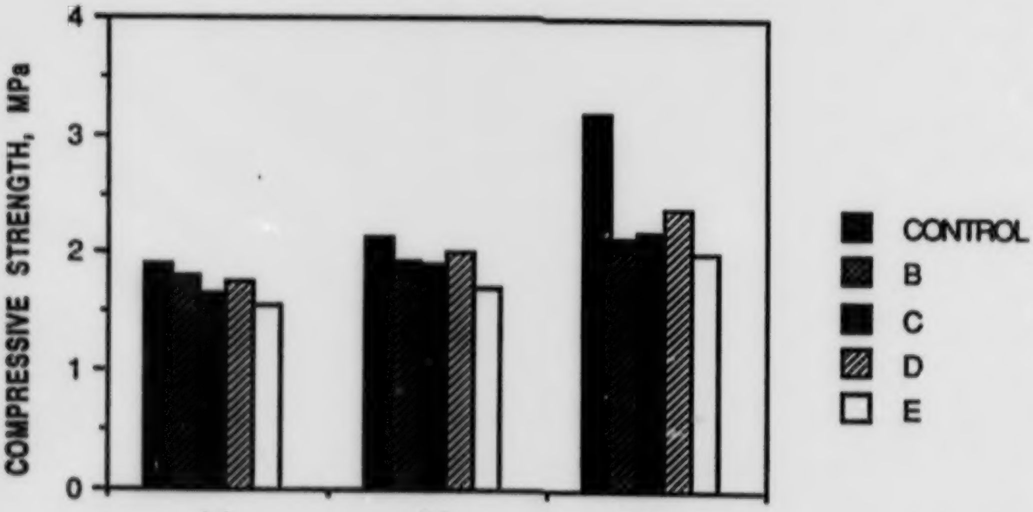
- Sets B and C indicated that the finer slag of 4000 Blaine did not contribute to the compressive strength.

- Set D, requenched lime-enriched slag, obtained the highest strength values in samples containing slag. The compressive strength of set D reached to about 75% of the control set A.

- The results indicated that copper slag did contribute to the strength development of P.C./ slag mixes. However, the use of accelerating agents to speed up the curing process should be investigated.



graph# 1 : RESULTS OF REPORT #1



graph #2 : RESULTS OF REPORT #1

REPORT # 2

CEMENT / COPPER SLAG CONCRETE CUBE TESTING RESULTS

OBJECTIVE: To study the cementitious property of copper slag . This is to investigate the rate of increase in compressive strength of the concrete cubes with different amounts of cement and/or copper slag.

PROCEDURE:

- Sample preparation followed the C 109-80 ASTM standard.
- Testing was carried out using blended Type 10 cement.
- KCM copper slag, 4000 Blaine, was used in this testing program.
- Each set was made of three 2-in. cubes cured for 28 days.
- In sets C and E, the sand/P.C. ratios were increased accordingly to have the desired cement concentration.

MATERIAL :

- The materials used in each set were:

SET A : (100% CEMENT)

P.C. = 400 gr.
slag = 0.0
water = 196
sand = 1100

SET B : (40% CEMENT/ 60% SLAG)

P.C. = 160 gr.
slag = 240
water = 180
sand = 1100

SET C : (40% CEMENT / NO SLAG)

P.C. = 240 gr.
slag = 0.0
water = 327
sand = 1650 (2.75/0.4 = 6.875 FOR RATIO OF SAND /P.C.)

SET D : (60% CEMENT / 40% SLAG)

P.C = 240 gr.
slag = 160
water = 180
sand = 1100

SET E : (60% CEMENT / NO SLAG)

P.C. = 360 gr.
slag = 0.0
water = 327
sand = 1649 (2.75/0.6 = 4.58 FOR THE RATIO OF SAND/P.C.)

RESULTS :

SET	COMPRESSIVE STRENGTH, MPa AT 28 DAYS
A	22.4
B	14.2
C	4.6
D	20.7
E	11.7

DISCUSSION:

The results indicated that copper slag had a moderate cementitious property if the replacement of the cement with slag did not exceed 40%. Replacement of 40% cement with slag, set D, decreased the compressive strength by just 8%. The samples containing slag will cure for a longer period of time and eventually would exceed the strength of the control set A.

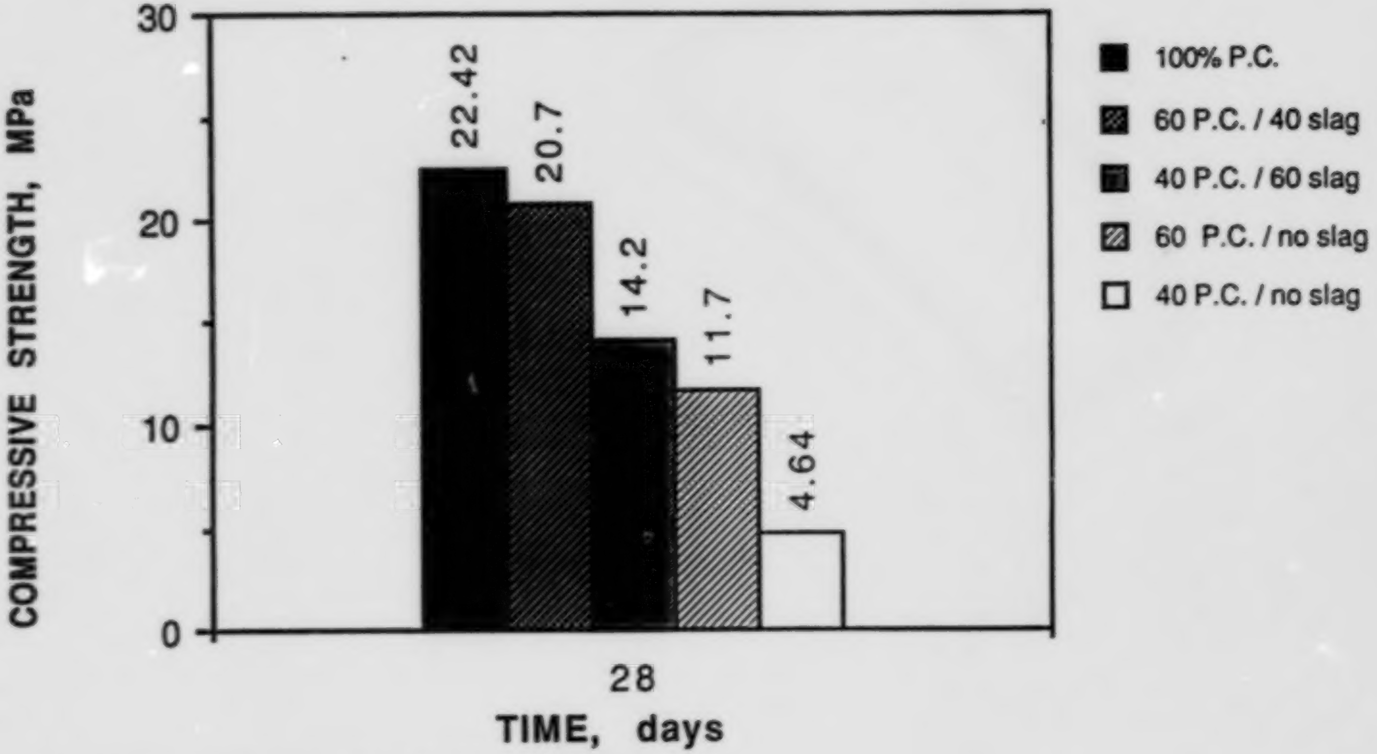
CONCLUSION :

- Set D with 40% replacement of cement with KCM copper slag obtained a strength of 20.7 MPa after 28 days which is 92% of the control set A.

- Since the slag curing process is much longer compared to the cement, the longterm strength of set D would surpass the strength of the control set A.

- An addtional 20% replacement of cement in set B decreased the compressive strength by 45% compared to set D.This indicates that the partial replacement of cement with slag may not exceed 40%.

- Sets C and E containing no slag and low cement content had very low compressive strengths. Comparison of these results with sets B and/or D showed that the slag had a major contribution in strength development of the samples.The high water to cement ratio and high sand to cement ratio might have caused the low compressive strength of sets C and E.



graph# 3 : RESULTS OF REPORT #2

REPORT # 3

USE OF ACCELERATORS TO INCREASE THE EARLY COMPRESSIVE STRENGTH, test #1

OBJECTIVE: To study the possibility of using accelerators such as $FeSO_4$ and K_2SO_4 to speed up the curing process of cement and slag mixes.

PROCEDURE:

- Sample preparation followed the C109-80 ASTM standard.
- Pulp density was 85%.
- Alluvial sand was used as the filler.
- Testing was carried out by using a point load testing machine.
- The mixed combinations are listed below :

SET A
alluvial sand = 952.54 gr.
P.C. = 95.25
slag = 0.0 **average comp. strength= 168 psi (1.16 MPa)**
accel. = 0.0
water = 181.4

SET B
alluvial sand = 952.54 gr.
P.C. = 57.20
slag = 38.10 **average comp. strength = 44.8 psi (0.31 MPa)**
accel. = 0.0
water = 181.40

SET C
alluvial sand = 952.54 gr.
P.C. = 57.20
slag = 38.10 **average comp. strength = 33 psi (0.23 MPa)**
$FeSO_4$ = 9.52 (10% OF BINDER)
water = 181.40

SET D
alluvial sand = 952.54 gr.
P.C. = 57.20
slag = 38.10 **average comp. strength = 30.4 psi (0.21 MPa)**
$FeSO_4$ = 19.05 (20% OF BINDER)
water = 181.40

SET E
- Same as set C but using K_2SO_4 as the accelerator.

average comp. strength = 118 psi (0.81 MPa)

SET F

- Same as set D but using K_2SO_4 as the accelerator.

average comp. strength = 88 psi (0.6 MPa)

NOTE:
Slag= Kidd Creek copper slag, about 3000 Blaine.
Cement= Type 10, 4 products blended.
Cement/slag ratio is kept constant at 60/40.

OBSERVATIONS:

- Sets D and F had orange coloured spots on the cube surfaces and this was probably due to high amount of accelerator used causing the possible chemical attacks.

- Set B had a very low compressive strength and it was difficult to take the cubes out of their moulds after 4 days of curing. The other sets were quite solid after the same time period.

RESULTS:

- All samples had a 6 day curing time.

- Result from set E was encouraging. The use of the accelerator had increased the compressive strength by 265% comparing to set B (having no accelerator) after 6 day curing, graph #4.

- Set F indicates that the amount of accelerator in the mix had passed the optimal percentage.

- Set B had hardly any compressive strength after being cured for 6 days.

- $FeSO_4$ accelerator did not increase the early strength of the mixes.

DISCUSSION:

More detailed study of the type and the performances of different accelerators should be carried out to verify their potential future use. The longterm effects should also be considered for material deterioration caused by chemical reactions of these accelerators. Other important factors for possible future use are the availability and the costs of these materials.

CONCLUSION:

- K_2SO_4 showed good reaction properties with the P.C./slag mixes.

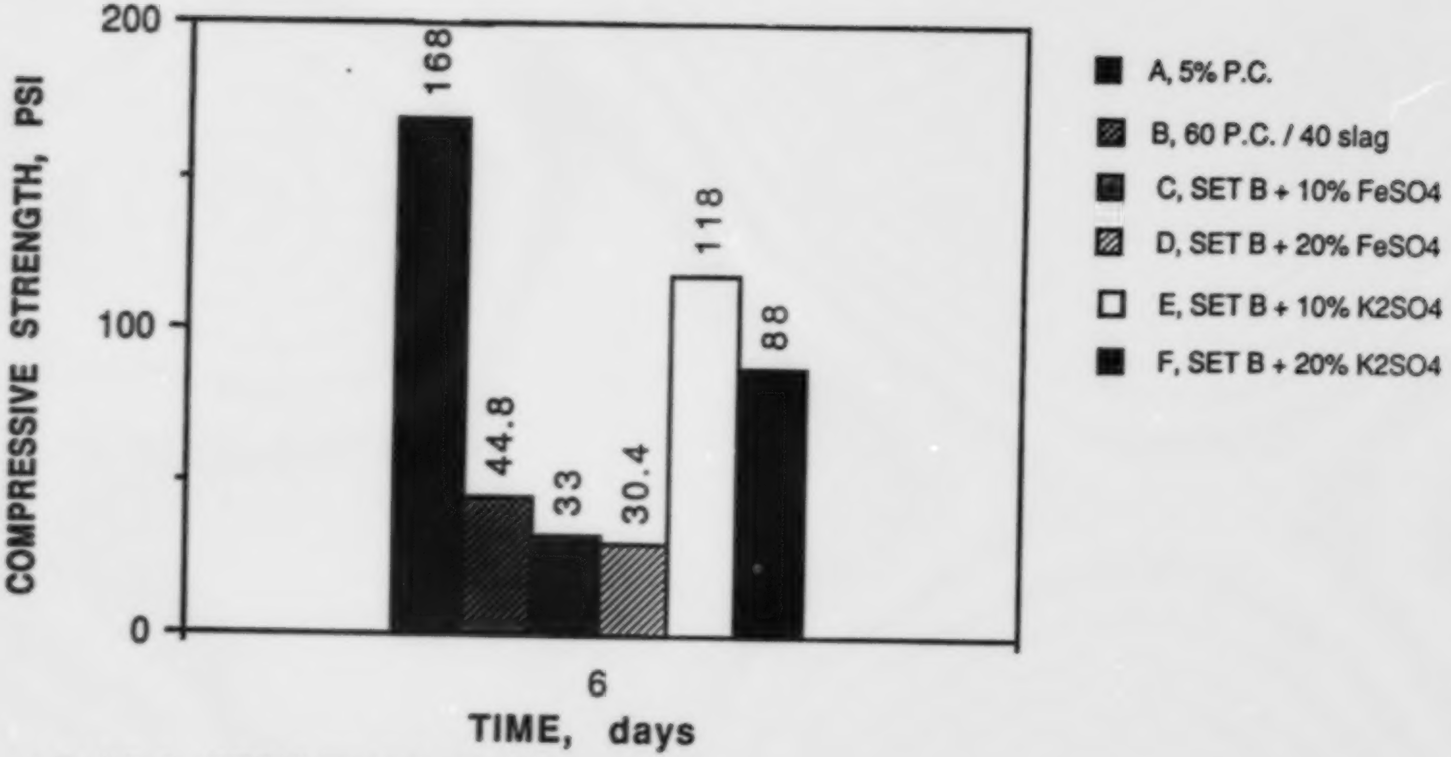
- $FeSO_4$ did not cause any increase in early strength. This accelerator is known to react slower than others, but in longterm it could gain a higher strength than the samples containing K_2SO_4.

RECOMMENDATIONS:

- Repeat the same set of tests to verify the obtained results and at the same time study the longterm effects of these accelerators on the compressive strength of the mixes.

- Since set F (20% accel.) showed lower strength than set E (10% accel.), it is believed that the amount of accelerator in set F has exceeded the optimal dosage. The amount of accelerator at 5% and 15% of the binding material should also be investigated.

- Test $FeSO_4$ with longer curing time, 14, 28 and 56 days.



graph# 4 : RESULTS OF REPORT #3.

REPORT # 4

CEMENT/SLAG COMPRESSIVE STRENGTH DEVELOPMENT USING ACCELERATORS, test #2

OBJECTIVE: To study the effect of different accelerators on the compressive strength development of P.C. /slag binder. The accelerator will be used to speed up the curing process which was slowed down by replacing 40% of the cement with copper slag. Accelerators used in this study were $FeSO_4$, K_2SO_4, $CaCl_2$ and Daracell, a product of W. R. Grace Chemical Company. Various amounts of these materials were tested to find the optimum percentage of the accelerator needed in each case.

PROCEDURE :

- All these tests were carried out using Kidd Creek regular copper slag, 3000 Blaine. Based on the final results of these tests, other slag sizes could be also tested.
- Sample preparation followed the C 39 - 83b ASTM standard.
- The materials used in each set were :

SET A: control		**TOTAL WEIGHT %**
agg.	= 5.8 kg (same for all sets)	62%
sand	= 2.8 kg (same for all sets)	30%
P.C.	= 0.435 kg (100% cement)	4.6%
water	= 0.357 kg	3.8%

SET B :

Same as set A, but 3% of total weight was cement, 260.7 gr. This was to see if Kidd Creek slag had any binding properties.

SET C :

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.260 kg, 3% of the sand and aggregate weight
slag = 0.173 kg, 2% of the sand and aggregate weight
water = 0.357 kg

SET D:

- Same as set C with addition of 5% of cementing material as K_2SO_4, 22.68 gr.

SETS E AND F:

- Same as set C with addition of 10 and 15% of cementing material as K_2SO_4 in sets E and F respectivley

SET E : 45.3 gr. SET F: 68.0 gr.

SETS G,H AND I :

- In this stage, the effect of 5, 10 and 15% addition of Daracell to the fill material was investigated.
- Unlike other material, this accelerator was in liquid form and after consulting with Mr. Jim Parson of Grace Chemical Company it was found that the solid content of this slurry was around 35% .This information was needed for keeping the moisture content as constant as possible in all sets.

SET G:
- liquid Daracell = 21.7 gr. (5%)
water = 343.3 gr.

SET H :
- liquid Daracell = 43.5 gr. (10%)
water = 329.1 gr.

SET I :
- liquid Daracell = 68.0 gr. (15%)
water = 315.05 gr.

SETS J AND K :
- In these two sets 5 and 10% $FeSO_4$ was added to set C:

SET J :
- $FeSO_4$ = 22.7 gr. (5%)

SET K :
- $FeSO_4$ = 45.36 gr. (10%)

SETS L AND M :
- Same as set C, but 3 and 5% $CaCl_2$ were added to the mix.

SET L :
- $CaCl_2$ = 13.6 gr. (3%)

SET M :
- $CaCl_2$ = 22.7 gr. (5%)

RESULTS:

- Table 3 presents the results obtained for each set at 28 days. The final results are also shown by graphs # 9 and #10 .

DISCUSSION :

- Daracell and K_2SO_4 performed well with P.C./slag mix and addition of Daracell increased the compressive strength even higher than the control mix . The extra cost of Daracell, however, could make it impractical for use in a backfill system. Potasium sulphate obtained a much higher compressive strength after 56 days of curing. Ferrous sulphate did not perform satisfactory after 56 days of curing .

CONCLUSION :

- Comparing sets B and C showed that the addition of slag to set C which had 3% cement content increased the compressive strength by 18%. This is 54% of the strength of the control set A, which had 5% cement content.

- Set H, 10% Daracell, showed the best results and achieved a 4% higher compressive strength than the control set A.

- There was a 51% increase in strength when the amount of Daracell was increased from 5 to 10%, sets G and H respectively. The strength decreased by 14% by increasing the Daracell from 10 to 15% as shown in graph #5.

- Sets D, E and F having 5, 10 and 15% K_2SO_4 showed a much better results after curing for 56 days compared to 28 day tests. Set D, 5% K_2SO_4, achieved 76% compressive strength of the control set, graph # 6.

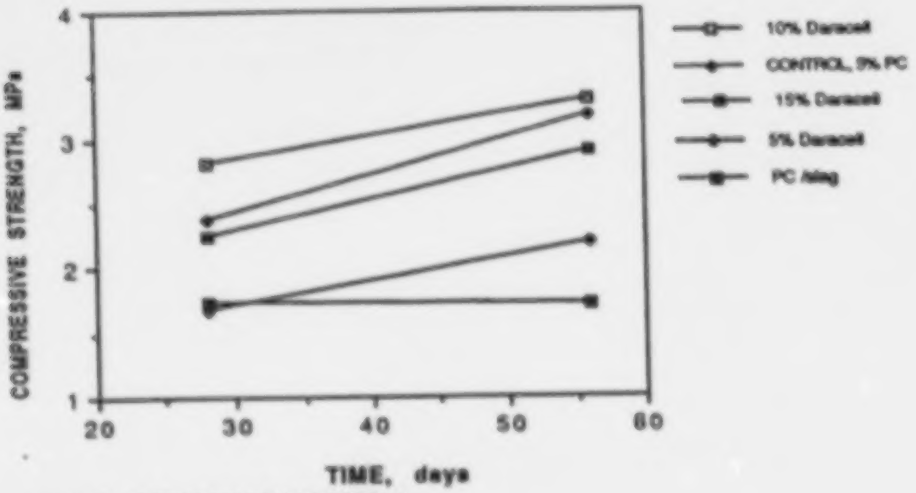
- Adding 5% $CaCl_2$, set M, to P.C./slag mix, set C, increased the strength by 16% as compared to 19% when 3% $CaCl_2$ was added, set L, graph #7.

- The strength of set M, 5% $CaCl_2$, was 76% of set H, 10% Daracell, and 90% of the control set A after 28 days curing. These values drop to 61% and 63% respectively after 56 days.

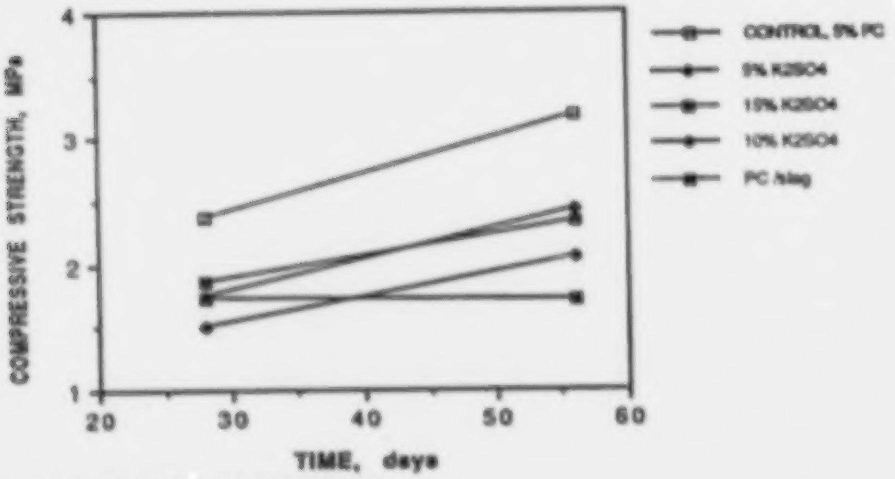
- Ferrous sulphate did not contribute in increasing the compressive strength of the mixes after 56 days of curing . It even decreased the strength when 5% $FeSO_4$ was used, graph # 8

RESULTS AFTER 28 and 56 DAYS OF CURING:

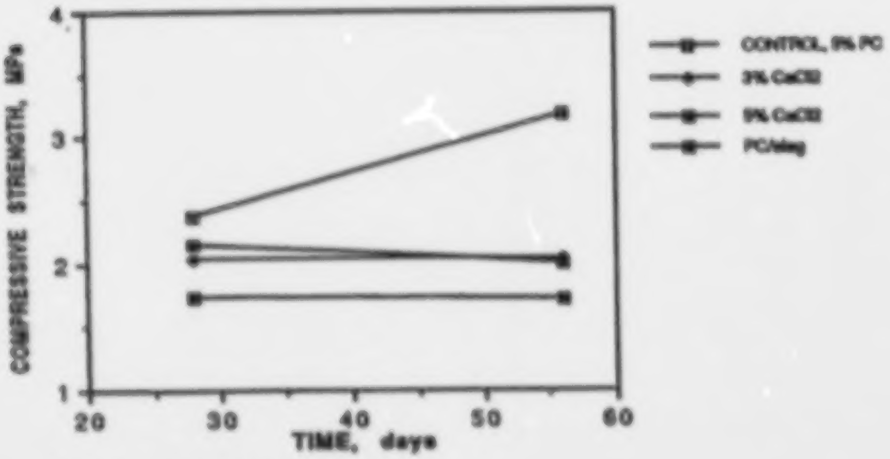
	COMPRESSIVE STRENGTH, MPa	
	28 days	56 days
SET A : CONTROL, 5% P.C.	2.37	3.18
SET B : 3% P.C., NO SLAG	1.14	1.46
SET C : 3% CEMENT + 2% SLAG , NO ACCELERATOR	1.73	1.72
SET D : ADDING 5% K_2SO_4	1.75	2.42
SET E : ADDING 10% K_2SO_4	1.5	2.05
SET F : ADDING 15% K_2SO_4	1.87	2.34
SET G : ADDING 5% DARACELL	1.68	2.18
SET H : ADDING 10% DARACELL	2.81	3.3
SET I : ADDING 15% DARACELL	2.24	2.9
SET J : ADDING 5% $FeSO_4$	1.22	1.42
SET K : ADDING 10% $FeSO_4$	1.77	1.70
SET L : ADDING 3% $CaCl_2$	2.04	2.04
SET M : ADDING 5% $CaCl_2$	2.14	2.0



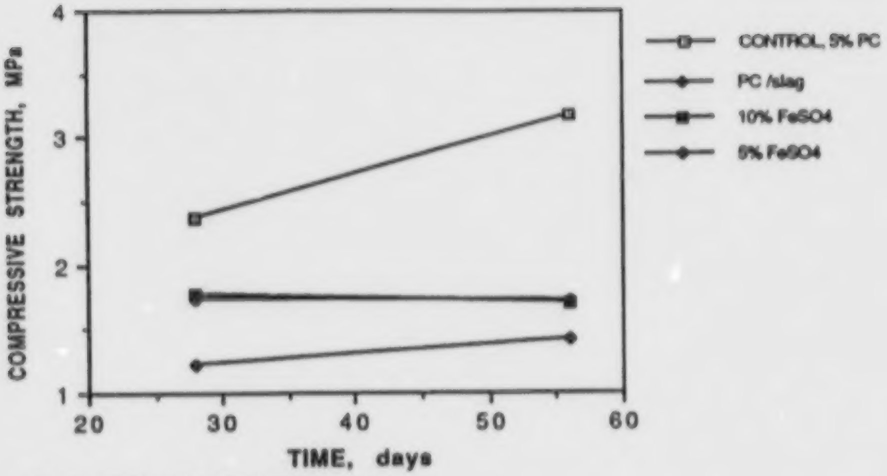
graph #5 : RESULTS OF REPORT #4



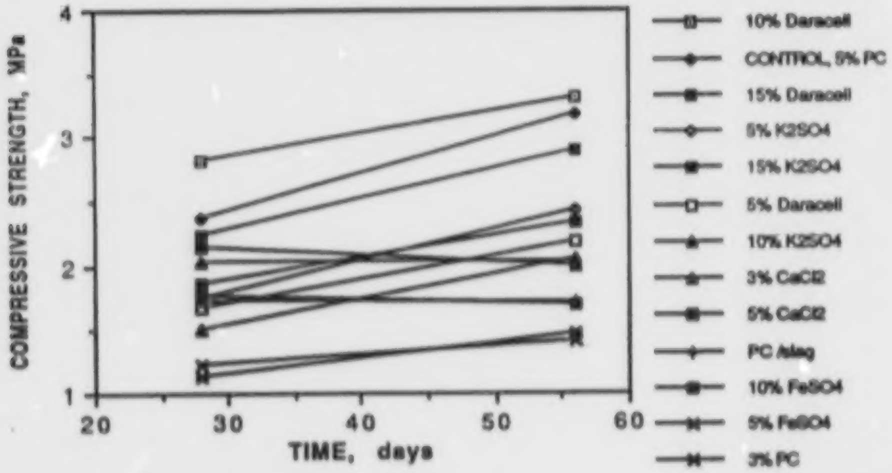
graph #6 : RESULTS OF REPORT #4



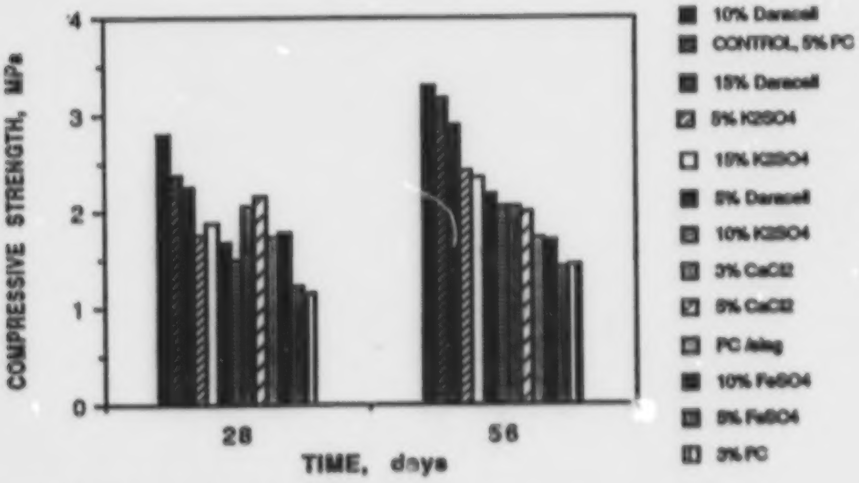
graph #7 : RESULTS OF REPORT #4



graph #8 : RESULTS OF REPORT #4



graph# 9 : RESULTS OF REPORT #4



graph# 10: RESULTS OF REPORT #4

REPORT # 5

EFFECT OF ACCELERATORS ON COMPRESSIVE STRENGTH DEVELOPMENT OF CEMENT/SLAG BINDER, test #3

OBJECTIVE : The same as Report # 4. The accelerators used in this study were Daracell, a product of Grace Chemical Company, Pozzolith 133-HE, a product of Master Builder chemical company, K_2SO_4 and $CaCl_2$. Various amounts of these materials were tested to find the optimum percentage of the accelerator needed in each case. These test results were then combined with the results of the the past tests to evaluate the performance of each accelerator.

PROCEDURE :

- Sample preparation followed the C39- 83b of ASTM standard.
- Each set was made of six 3-in. diam. cylinders with testing intervals of 28 and 56 days
- All these tests were carried out using Kidd Creek regular copper slag, 3000 Blaine.
- The materials used in each set were :

SET A : CONTROL

agg. = 5.8 kg (same for all sets)
sand = 2.8 kg (same for all sets)
P.C. = 0.435 kg
water = 0.357 kg

SET B : 40 SLAG/60 CEMENT

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.265 kg, 3% of the sand and aggregate weight
slag = 0.173 kg, 2% of the sand and aggregate weight
water = 0.357 kg

SET C : 7.5% DARACELL

- Same as set B with addition of 7.5% of cementing agents with Daracell.
liquid Daracell = 32.59 gr water = 336.2 gr

SET D: 12.5% DARACELL

- Same as set B with addition of 12.5% of Daracell.
liquid Daracell = 54.3 gr water = 322.11 gr
note: Results for 5, 10 and 15% Daracell concentrations are already available . The past results should not be compared with the present results due to difference in mixing procedure. Solid concentration of Daracell is around 35%.

SETS E AND F : 5 AND 10% POZZOLITH 133-HE

- Same as set B with addition of 5 and 10% of Pozzolith 133-HE
SET E:
liquid Pozzolith 133-HE = 21.73 gr water= 343.95 gr
SET F :
liquid Pozzolith 133-HE = 43.46 gr water = 330.5 gr
note : Solid concentration of the product is around 38%.

SET G AND H : 2 AND 7% $CaCl_2$

Same as set B with addition of 2 and 7% $CaCl_2$

SET G :

- $CaCl_2$ = 8.7 gr

SET H :

- $CaCl_2$= 30.42 gr

NOTE : Results for 3 and 5% concentrations were already available.

SET I AND J : 2 AND 4% K_2SO_4

Same as set B with addition of 2 and 4% K_2SO_4.

SET I :

- K_2SO_4 = 8.7 gr

SET J :

- K_2SO_4 = 17.4 gr

NOTE: Results for 5, 10 and 15% concentrations were already available.

RESULTS:

- Table 4 presents the results obtained for each set at 28 and 56 days.

COST ANALYSIS :

- Material costs :
  - P.C. = $100/ton
  - Flyash (C) = $60/ton
  - Blended flyash/cement = $ 80/ton
  - Daracell = $0.51/Litre , $0.23/lb
  - Pozzolith 133-HE =$1.0/Litre , $0.45/l
  - $CaCl_2$ = $380/ ton
- Average cement content in KCM backfill is around 5%,
  - 0.05 * 2000 = 100 lb of cementing binder/ ton of CRF
  - 0.05 * 100 = $5/ton of CRF (if 100% P.C. is used)
  - 0.05 * 80 = $4/ton of CRF (if blended flyash/cement is used)

NAME	CONC	COST	SAVINGS TO HANDLE ROCKFILL	
	%	($ / TON CRF)	100% PC	BLENDED
$CaCl_2$	2	0.38	31.0	61.0
	3	0.57	21.5	51.5
	5	0.95	2.5	32.5
	7	1.33	-16.5	13.5
DARACELL	5	1.15	-7.5	22.5
	10	2.3	-65.0	-35.0
	12.5	2.8	-94.0	-63.8
	15	3.45	-122.5	-92.5
POZZOLITH 133-HE	5	2.27	-63.5	-33.5
	10	4.55	-177.5	-147.5

SAMPLECALCULATION

For 2% $CaCl_2$
cost per ton of $CaCl_2$ = \$380 which is equal to \$0.19/lb.
2000 * 0.05= 100 lb of cementing material /ton of CRF
0.02 * 100 = 2 lb of $CaCl_2$
2 * \$0.19 = \$0.38/ton of CRF
P.C. COST + ALLOWABLE COST FOR SLAG + CHEMICAL COST = \$4 (cost of present CRF)
0.03*100 (or 80) +0.02* X + 0.38 = 4
X = \$31/ton of CRF or X= \$61/ton of CRF (IF BLENDED CEMENT/FLYASH IS USED)X= ALLOWABLE COST PER TON OF CRF FOR GRINDING AND TRANSPORTING THE SLAG.

DISCUSSION :

All the results were compared with control set B, 60 P.C./40 slag mix. Daracell and Pozzolith 133-HE improved the strength of P.C./slag mixes considerably but the cost of these materials were high and no savings could be obtained from using them in the fill system. 2% $CaCl_2$ increased the compressive strength by 20% compared to set B after 28 days of curing but the 56 day results were almost the same as the control set with no $CaCl_2$. This indicated that the $CaCl_2$ increases the curing rate in the first month and could be employed to get shorter mining cycle. K_2SO_4 showed a very disappointing result and did not improve the compressive strength of the fill material. The longterm results of both 2 and 4% K_2SO_4 were lower than the control set B and no benefit will be gained by using this material.

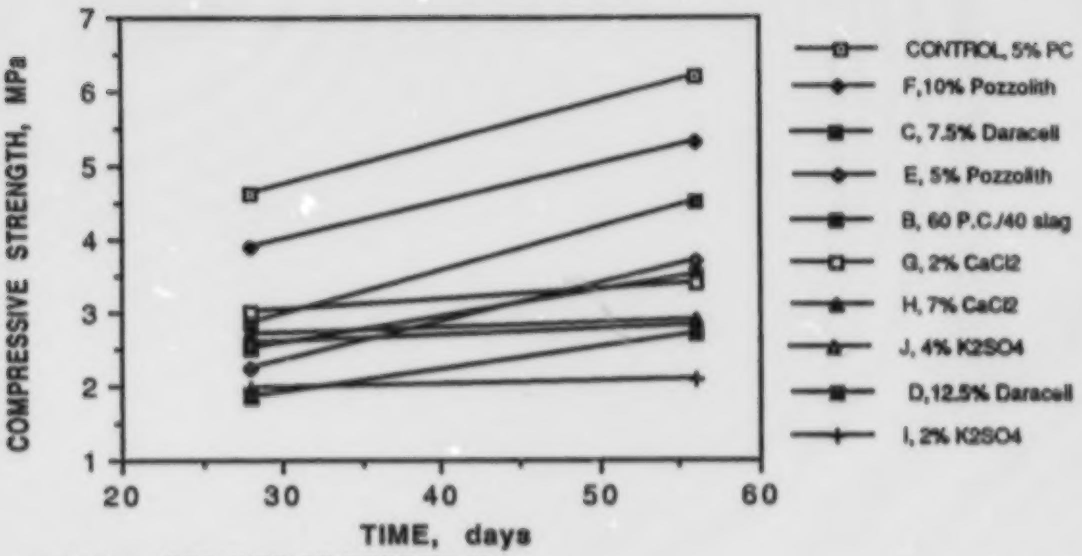
CONCLUSION :

- Graphs #13 & 14 indicated that the optimum percentage of Daracell in the mix was around 10%.
- 7.5% Daracell solution in the mix, set C, increased the strength by 28% compared to the control set B. The increase in concentration from 7.5 to 12.5% decreased the strength by 40%, graphs # 13 & 14.
- Since two different mixing methods were employed, the normalized strength increase rates for each chemical compared to 60 P.C./ 40 slag control mix are presented in graphs 13-19.
- The strength of set G, 2% $CaCl_2$, was 64% of set F, 10% Pozzolith 133-HE. Graphs #15 & 16 showed that the optimum $CaCl_2$ concentration in the mix was around 3%.
- 2 and 4% K_2SO_4 decreased the compressive strength of the samples considerably after 28 and 56 days of curing, graphs 17 & 18.
- Set F, 10% Pozzolith 133-HE, reached 85% strength of the control set A, 5% P.C., after 56 days of curing. This set had a 51% higher strength compared to the control set B after 56 days of curing, graphs # 19 & 20.

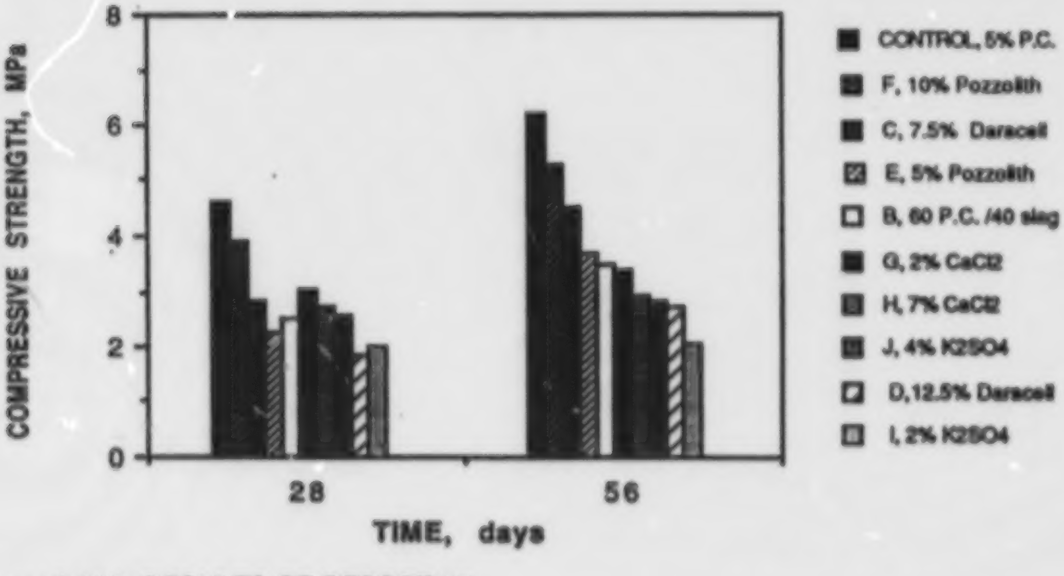
SET	COMPRESSIVE STRENGTH, MPa	
	28 days	56 days
A, CONTROL	4.6	6.2
B, 60 CEMENT/ 40 SLAG	2.5	3.5
C, 7.5% DARACELL	2.83	4.5
D, 12.5%	1.86	2.7
E, 5% POZZOLITH 133-HE	2.25	3.7

F, 10%	3.90	5.3
G, 2% $CaCl_2$	3.01	3.4
H, 7% $CaCl_2$	2.71	2.9
I, 2% K_2SO_4	1.98	2.06
J, 4% K_2SO_4	2.56	2.81

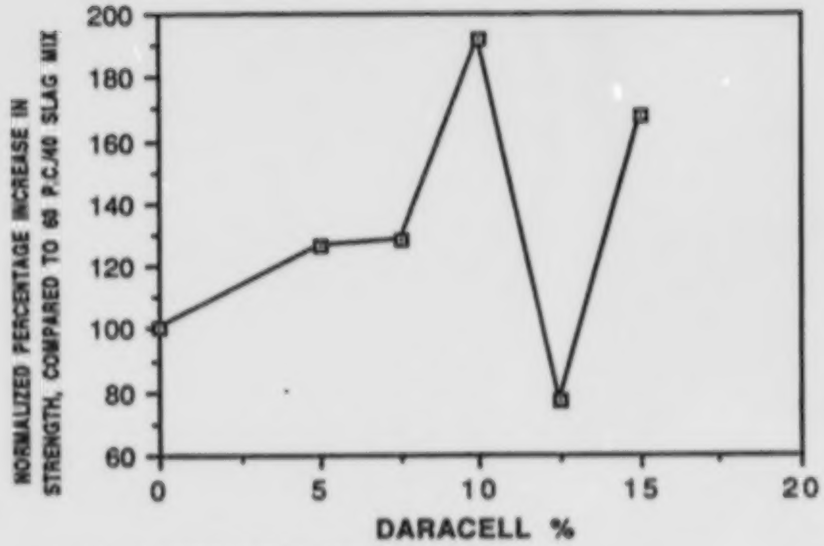
TABLE 4: 28 and 56 DAY RESULTS OF ALL DIFFERENT CONCENTRATIONS



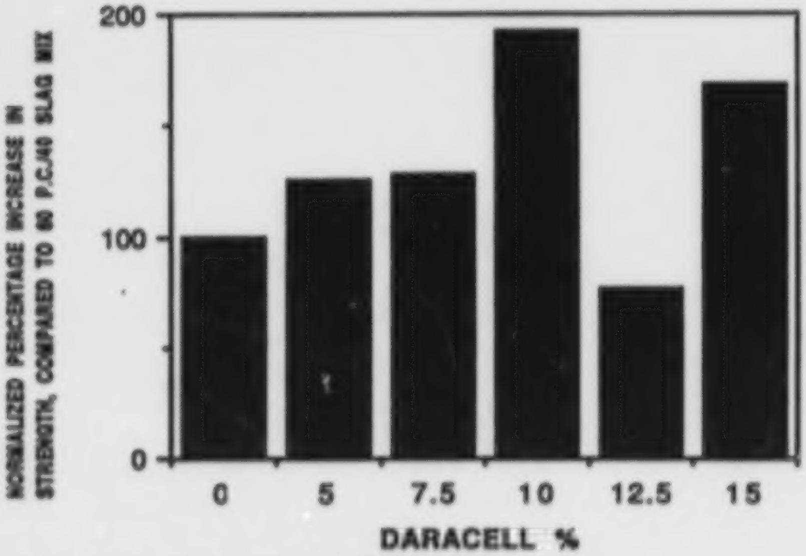
graph #11 : RESULTS OF REPORT #5



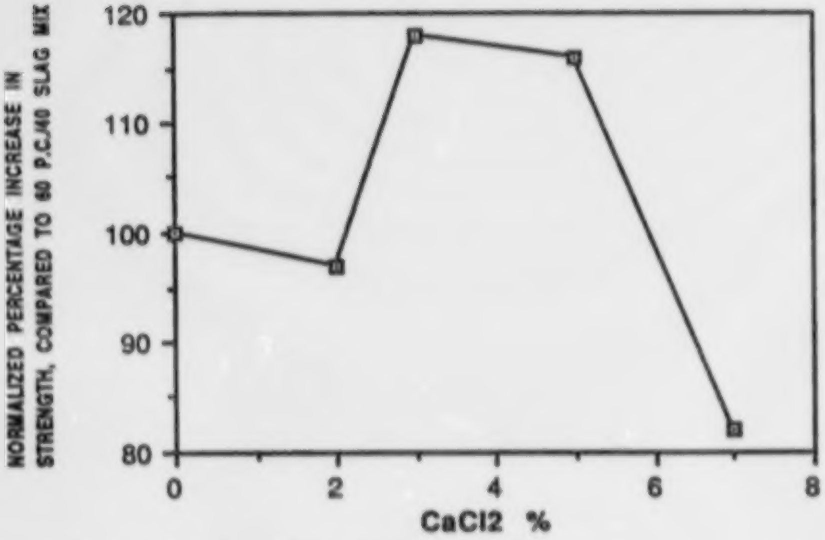
graph #12: RESULTS OF REPORT #5



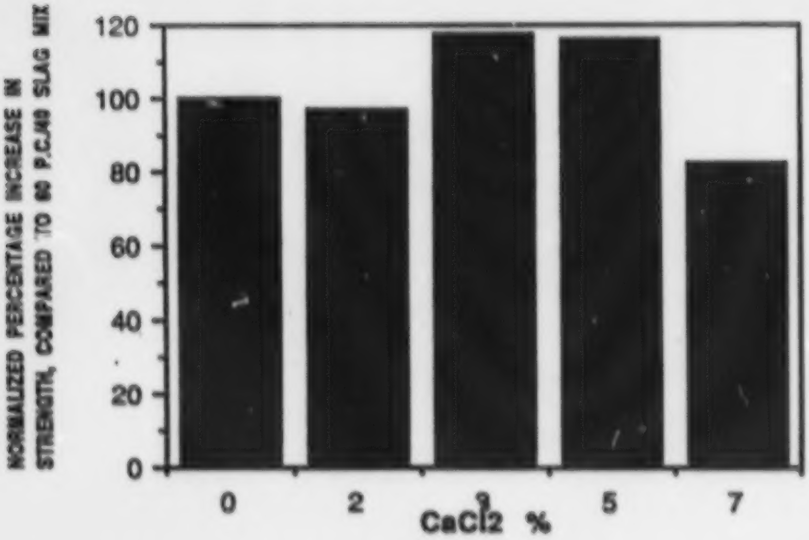
graph #13: RESULTS OF REPORT #5



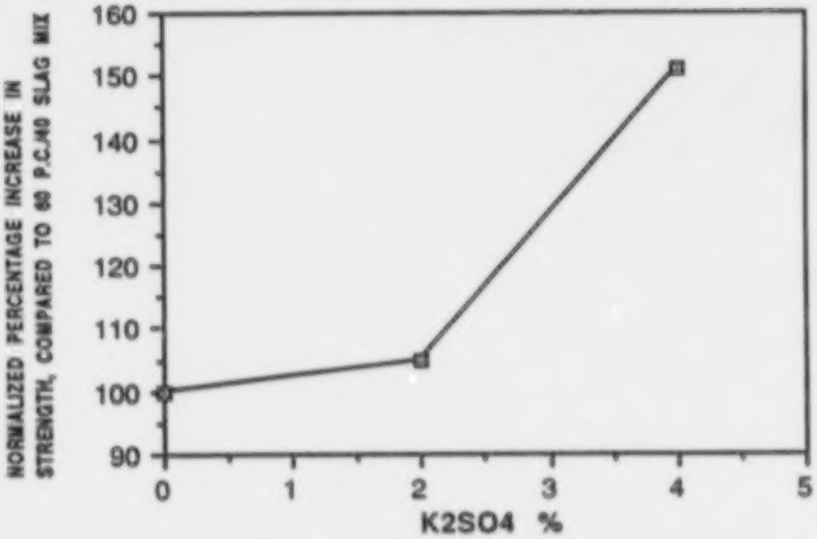
graph #14 : RESULTS OF REPORT #5



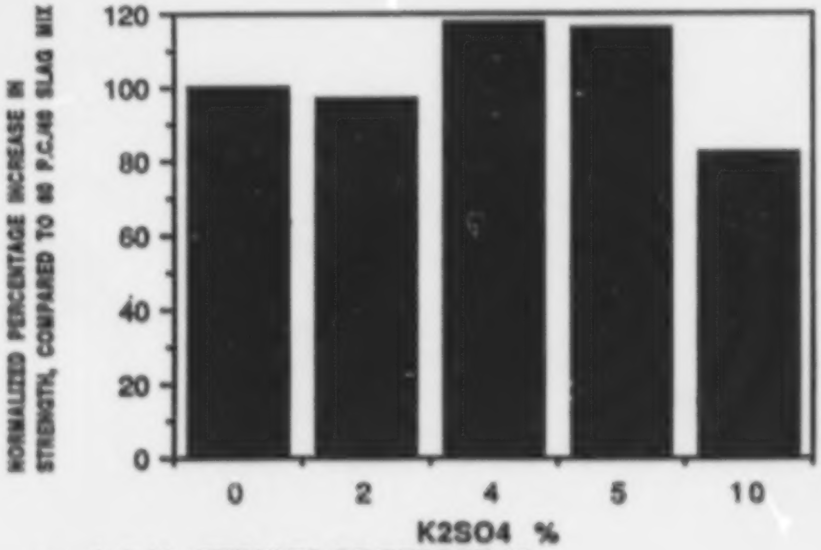
graph # 15 : RESULTS OF REPORT #5



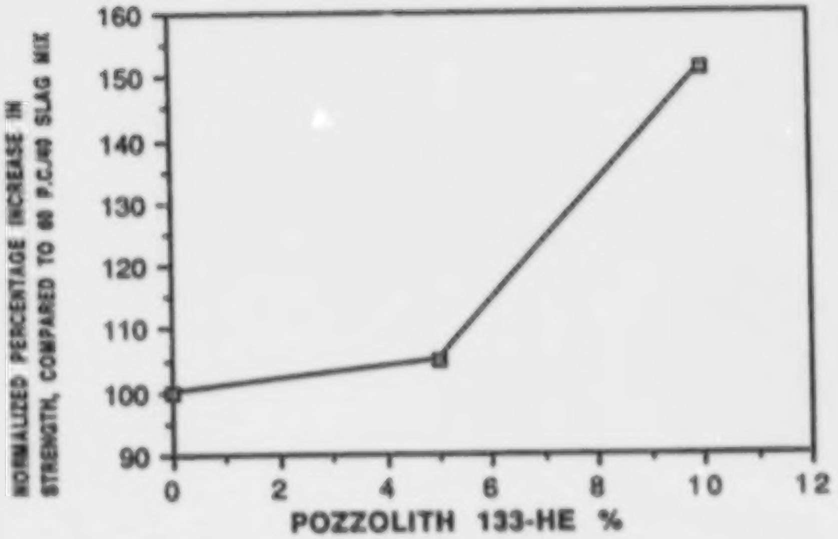
graph # 16: RESULTS OF REPORT #5



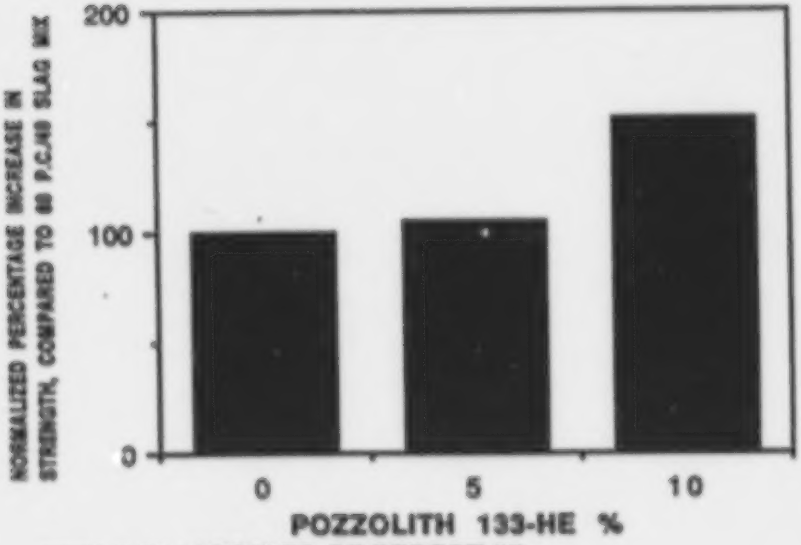
graph # 17 : RESULTS OF REPORT #5



graph # 18 : RESULTS OF REPORT #5



graph # 19 : RESULTS OF REPORT #5



graph # 20 : RESULTS OF REPORT #5

REPORT # 6

EFFECT OF ACIDULATED TYPE F FLYASH ON BACKFILL STRENGTH

OBJECTIVE : This study involves the partial replacement of P.C. with Type F flyash. Flyash normally has high iron and/or aluminium content. By treating the flyash with strong mineral acids the excess metals could be removed. The goal was to convert the oxides of excess metals into soluble salts that could be washed away. Removal of the undesirable metals create a much greater surface area for the reaction of silica. Hydrochloric and Sulphuric acids at two different concentrations were selected for testing.

PROCEDURE :

- All tests were carried out using blended Type 10 P.C., and Type F flyash from a thermal generating station near Toronto, Ontario.
- Sample preparation was the same as C 39-83b ASTM.
- Each set was made of six 3-in. diam. cylinders with curing time of 28 and 56 days.
- Acids weighing 1.2 times the weight of the flyash were used to acidulate the flyash. The acidulated solution was allowed to chemically react for 15 and/or 60 minutes before mixing with the rest of the materials. Before using the acidulated flyash in the final mix, the acid on top of the mix was decanted. The amount of acid left in the mix was then calculated, and it effected the final amount of water to be added to the mix. It should be noted that the moisture content in acidulated samples was about 1% higher than the one in unacidulated sets, likely causing the decrease of the strength in the acidulated sets.

- The materials used in each set were :

SET A :			**TOTAL WEIGHT %**
agg.	= 5.8	kg	62.0
sand	= 2.8		30.0
P.C.	= 0.435		4.6
flyash	= 0.0		
acids	= 0.0		
water	= 0.357		3.8

aggregate and sand weigh the same for all the following sets.

SET B : (33% P.C., 66% flyash)

P.C. = 0.144 kg
flyash = 0.29
acids = 0.0
water = 0.357

P.C. and flyash are the same for the rest of the mixes .

SET C : (38% HCL, 60 MINUTES)

acid = 0.35 kg (138 gr. was decanted)
water = 0.257 kg (Moisture content was kept around 5% for the rest of the sets)

SET D : (38% HCL, 15 MINUTES)

acid = 0.35 kg (140 gr. was decanted)
water = 0.259 kg

SET E : (20% HCl, 60 MINUTES)
acid = 0.35 kg (148 gr. was decanted)
water = 0.26 kg

SET F : (20% HCl, 15 MINUTES)
acid = 0.35 kg (162 gr. was decanted)
water = 0.274 kg

SET G : (38% H_2SO_4, 60 MINUTES)
acid = 0.35 kg (103 gr. was decanted)
water = 0.223 kg

SET H : (38% H_2SO_4, 15 MINUTES)
acid = 0.35 kg (110.8 gr. was decanted)
water = 0.23 kg

SET I : (20% H_2SO_4, 60 MINUTES)
acid = 0.35 kg (133 gr. was decanted)
water = 0.253 kg

SET J : (20% H_2SO_4, 15 MINUTES)
acid = 0.35 kg (147 gr. was decanted)
water = 0.267 kg

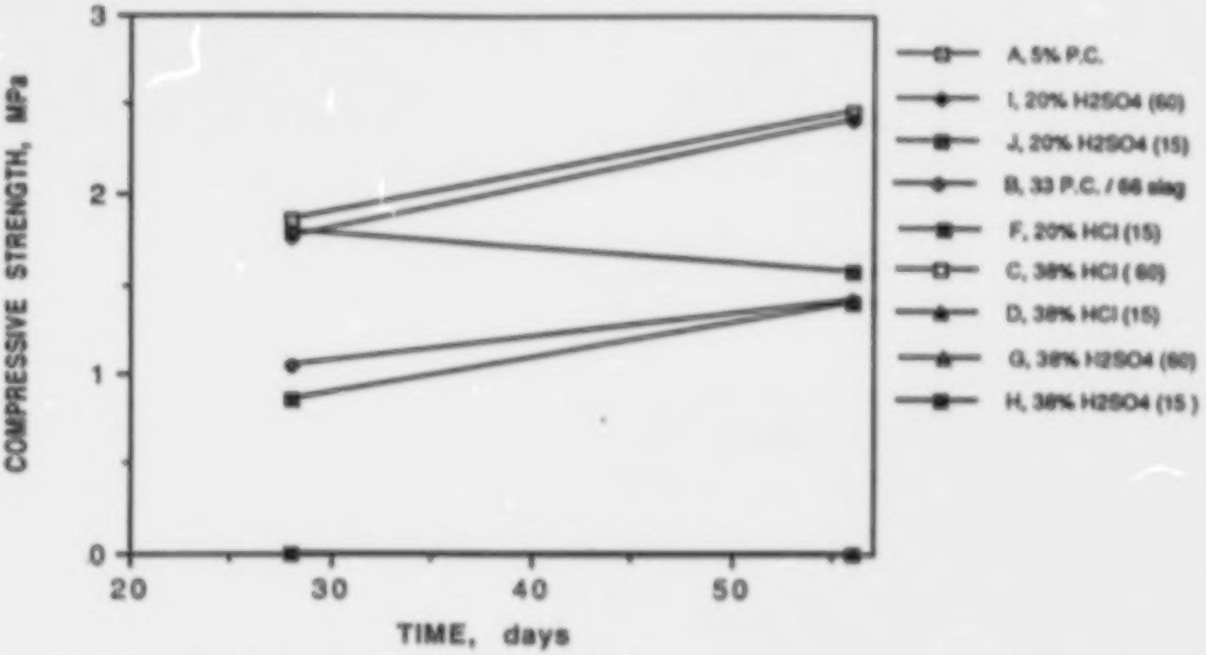
RESULTS : (graphs # 21 & 22)

DAYS	A	B	C	D	E	F	G	H	I	J
28 (MPa)	1.86	1.05	0.0	0.0	0.39	0.85	0.0	0.0	1.76	1.8
56 (MPa)	2.46	1.42	0.0	0.0	0.67	1.4	0.0	0.0	2.4	1.57

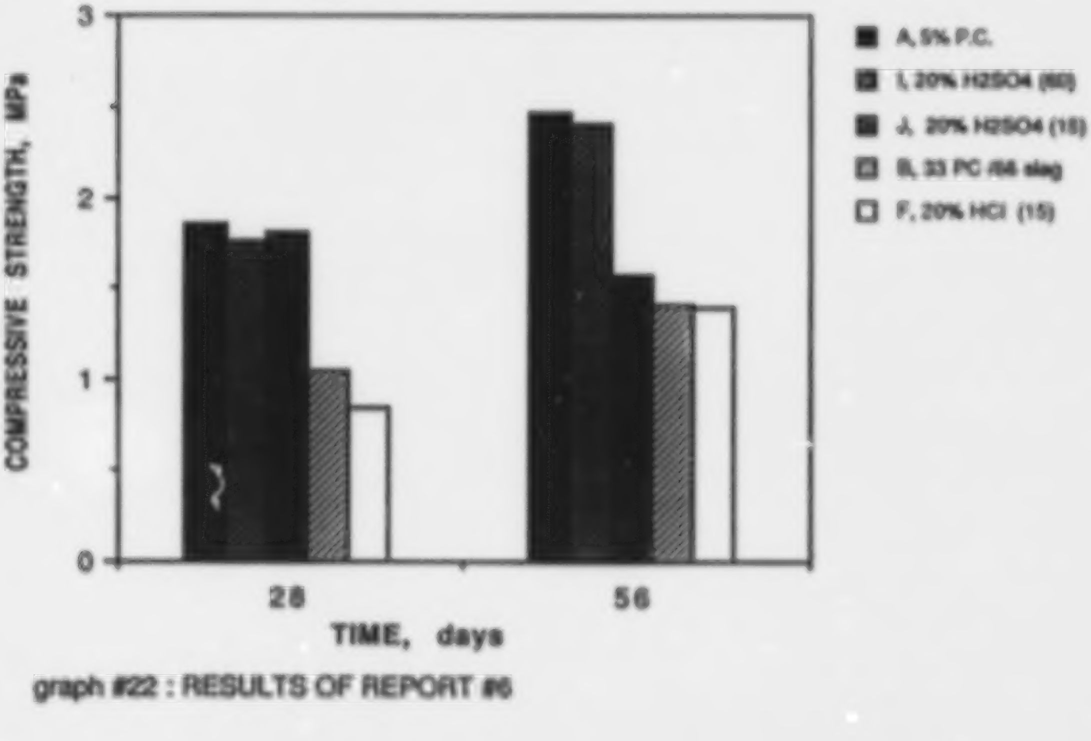
DISCUSSION :

- Acidulation of Type F flyash with HCl did not show any merit.

- Acidulated flyash with 20% H_2SO_4, 60 minutes, increased the strength by about 70%, compared to set B which was unacidulated. However, a great care must be taken in handling of the acid slurry during the preparation work.



graph #21 : RESULTS OF REPORT #6



graph #22 : RESULTS OF REPORT #6

REPORT # 7

ADDITION OF ACIDULATED TYPE F FLYASH TO THE KCM BACKFILL, test#2

The same as Report #6. Tests were carried out to study the effect of acidulated Type F flyash and ground copper slag on cementation development.

PROCEDURE :

- All tests were carried out using blended Type 10 P.C. and Type F flyash which was obtained from a thermal generating station near Toronto, Ontario. Copper slag of 3000 Blaine was used in these tests.
- Sample preparation was the same as C39-83b ASTM.
- Each set was made of six 3-in. diam. cylinders with curing time of 28 and 56 days.
- Acids weighing 1.2 times the weight of the flyash or slag were used to acidulate the cementing materials. The acidulated solution was allowed to chemically react for 30 minutes before mixing with the rest of the materials. Before using the acidulated flyash or slag in the final mix, the acid on top of the mix was decanted. The amounts of acid left in the mix was then calculated and the final amount of water to be added to the mix was determined. Unlike the testing program of report #6, the moisture content was kept constant throughout the entire testing program.

- The materials used in each set were :

SET A :			**TOTAL WEIGHT %**
agg	= 5.8	kg	62.0
sand	= 2.8		30.0
P.C	= 0.435		4.6
flyash	= 0.0		
acid	= 0.0		
water	= 0.357		3.8

aggregate and sand weigh the same for all the following sets.

SET B : (33% P.C., 66% flyash)
P.C = 0.144 kg
flyash = 0.29
acid = 0.0
water = 0.357

P.C. and flyash are the same for the rest of the mixes .

SET C : (60% P.C. / 40% SLAG)
P.C. = 0.261 kg
slag = 0.174
acid = 0.0
water = 0.357

SET D : (20% H_2SO_4, 30 MINUTES), SAME AS SET B + ACID
H_2SO_4 = 0.35 kg (107 gr. was decanted)
water = 0.114

SET E : (20% H_2SO_4, 30 MINUTES), SAME AS SET C + ACID

H_2SO_4 = 0.25 kg (177.4 gr. was decanted)
water = 0.284

SET F : (10% H_2SO_4, 30 MINUTES), SAME AS SET B + ACID
H_2SO_4 = 0.35 kg (136 gr. was decanted)
water = 0.143

SET G : (10% H_2SO_4 , 30 MINUTES), SAME AS SET C + ACID
acid = 0.25 kg (178 gr. was decanted)
water = 0.285

RESULTS :

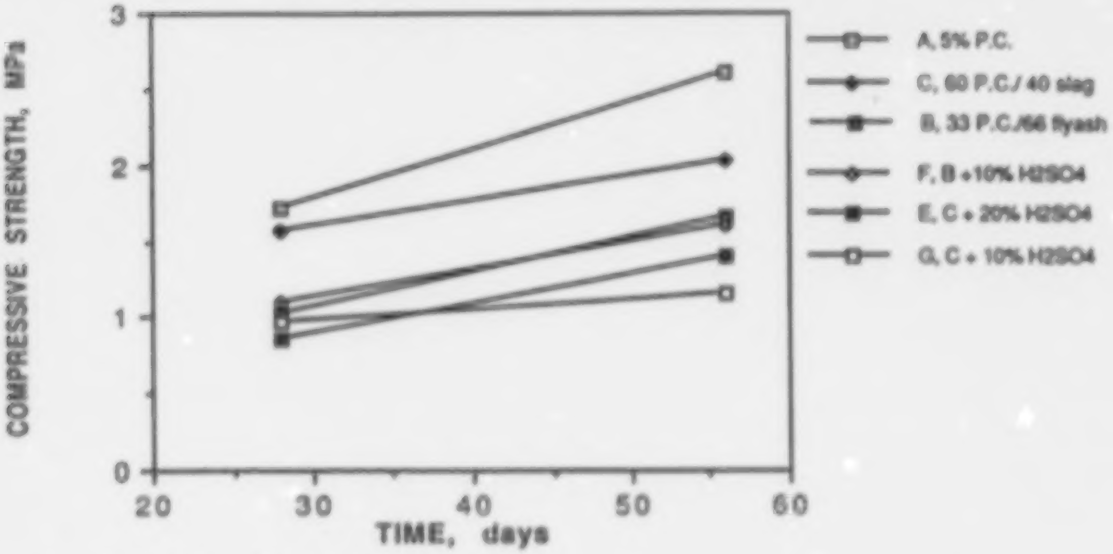
DAYS	A	B	C	D	E	F	G
28 (MPa)	1.72	1.03	1.57	1.16	1.42	1.1	0.96
56 (MPa)	2.6	1.66	2.03	N/A	1.41	1.6	1.15

DISCUSSION :

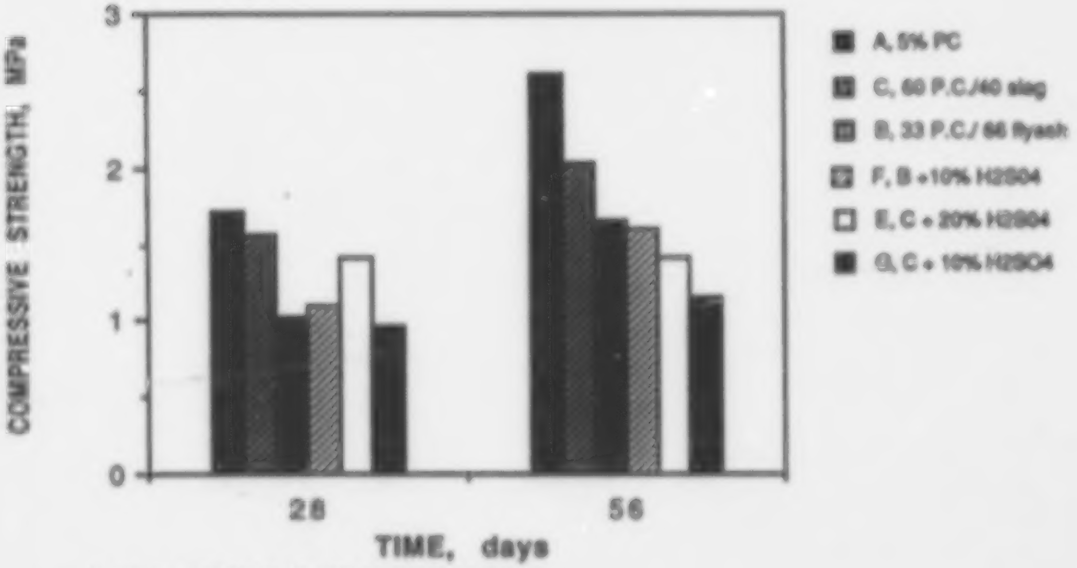
The acidulated flyash caused only a very small increase in the strengths of the P.C./flyash mixes, and the acidulated P.C./slag mixes showed negative results. This could have been due to the short acidulation time.

CONCLUSION :

- Acidulation of flyash with H_2SO_4 solution (20 and 10% concentrations) for a time period of 30 minutes did not contribute to the compressive strength development of the samples as much as the ones cast in Report #6.
- Both 10 and 20% addition of H_2SO_4 to the KCM copper slag decreased the compressive strength by 44 and 31% respectively, compared to unacidulated sample of set C.
- Acidulation of slag does not look promising according to the results obtained from this testing program.



graph #23 : RESULTS OF REPORT #7



graph #24 : RESULTS OF REPORT #7

REPORT # 8

STRENGTH INCREASE BY USING ADDITIONAL COPPER SLAG

OBJECTIVE : To study the cementitious property of KCM copper slag. This is to investigate the rate of increase in compressive strength of 3-in. diam. cylinders with different amounts of copper slag with constant Portland cement content.

MATERIAL :

- The materials used in each set were :

SET A : (100% P.C.), 5% cement

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.435 kg
slag = 0.0
water = 0.357 kg

SET B: (60 P.C. / 40 SLAG(1)), 3% P.C. and 2% slag

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.260 kg
slag = 0.173 kg
water = 0.357 kg (w/c =0.82)

SET C: (60 P.C. / 40 SLAG (2)), 3% P.C. and 4% slag

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.260 kg
slag = 0.347 kg
water = 0.500 kg (w/c = 0.82)

SET D : (60 P.C. / 40 SLAG (3)), 3% P.C. and 6% slag

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.260 kg
slag = 0.521 kg
water = 0.643 kg. (w/c = 0.82)

SET E : (60 P.C. /40 SLAG (4)), 3% P.C. and 8% slag

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.260 kg
slag = 0.695 kg
water = 0.786 kg (w/c = 0.82)

SET F : Same as set D with 20% less water, w/c=0.675

SET G : Same as set E with 20% less water, w/c= 0.675

PROCEDURE :

- Testing was carried out using blended Type 10 P.C.
- KCM copper slag, 3000 Blaine, was used in this testing program.
- Each set was made of six 3-in. diam. cylinders cured for 28 and 56 days.

RESULTS :

	COMP.STRENGTH	(MPa)
SET	28 DAYS	56 DAYS
A	2.76	3.02
B	1.80	2.30
C	2.40	2.65
D	2.24	2.40
E	1.20	1.73
F	2.65	3.63
G	1.70	2.04

DISCUSSION :

- Set F, had almost the same 28-day compressive strength compared to the control set A. The 56 day compressive strength of set F surpassed the strength of set A by 20%.

- 40% replacement of cement with slag decreased the compressive strength by 35 and 24% after 28 and 56 days of curing respectively, by comparing sets A & B.

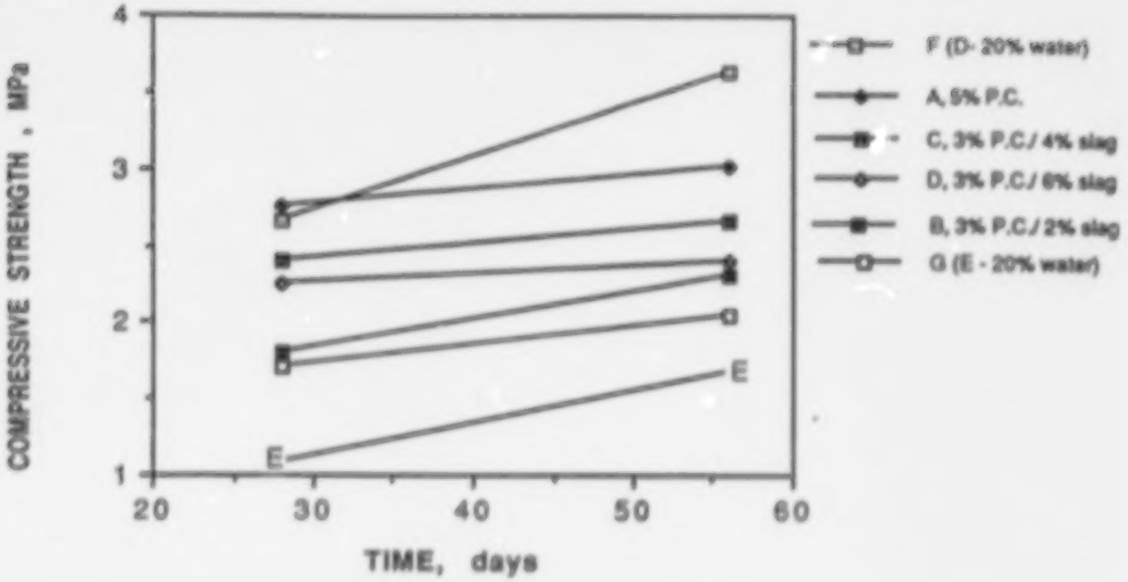
- Adding more slag in sets D and E did not contribute to the compressive strength, possibly due to the increased amount of water in the final mixes. Although water / binder ratio was kept the same as sets A, B and C, the mixes for sets D and E appeared soupy with approximately 20% excess water in the final mixes.

- Set F was the same mix as set D except for having 20% less water. This resulted in a strength increase of 51% after 56 days of curing, by comparing sets D and F.

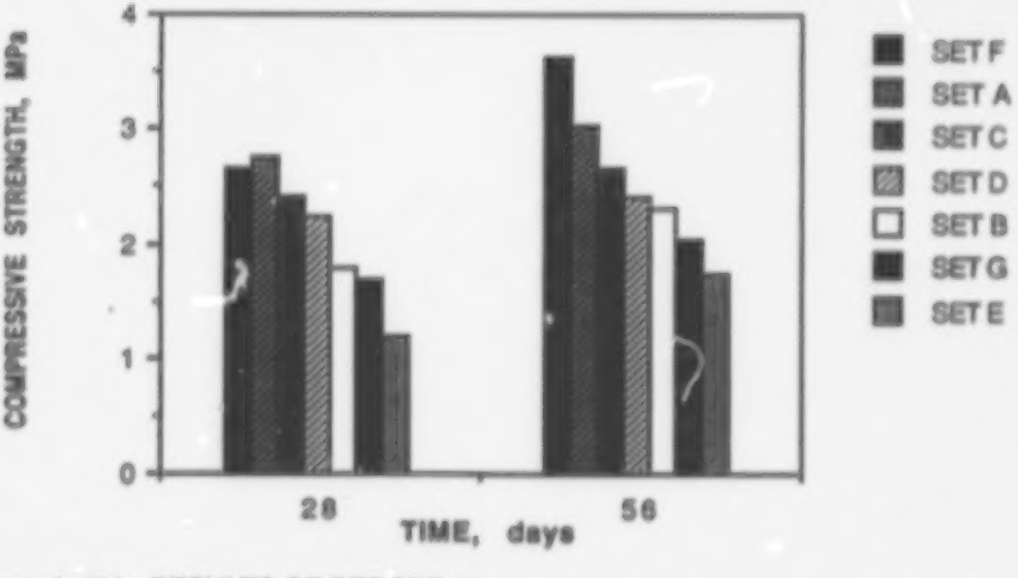
- Although set G had 20% less water than set E, it still appeared to have excessive water in the final mix. The excessive water in the mix prevented the development of the optimal strength of set G.

CONCLUSION :

- The results indicated that the KCM copper slag has a moderate cementitious property if the replacement of the cement with slag was below 40%. The compressive strength of the backfill could be increased by increasing the slag/binder ratio in the final mix. The testing indicated that an optimum compressive strength was obtained when the amount of copper slag was twice the amount of P.C. and by reducing water quantity, as shown in set F.



graph #25 : RESULTS OF REPORT #8



graph #26 : RESULTS OF REPORT #8

REPORT # 9

ADDITION OF LIME, GYPSUM AND SODIUM SILICATE TO KCM BACKFILL

OBJECTIVE: To investigate the effect of addition of lime, gypsum and sodium silicate to increase the early compressive strength of P.C./ flyash mixes. Addition of the above materials to P.C./copper slag binder was also investigated.

PROCEDURE :

All tests were carried out using bleaded Type 10 P.C. and Type C flyash. KCM copper slag of 3000 Blaine was used in these tests.

Each set was made of six 3-in. diam. cylinders to be tested at 28 and 56 day time intervals. The materials used in these sets were:

SET A: CONTROL, 100% P.C.

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.435 kg
water = 0.357 kg

SET B: CONTROL (P.C. AND SLAG)

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5 %)
slag = 0.436 kg (5%)
water = 0.366 kg

SET C: CONTROL (P.C. AND FLYASH)

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5%)
flyash = 0.218 kg (2.5%)
water = 0.357

SET D: set B + lime & gypsum #1

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5%)
slag = 0.436 kg (5%)
gypsum = 0.092 kg (1%)
lime = 0.06 kg (0.65 %)
water = 0.372 kg

SET E : set B + lime & gypsum #2

agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5%)
slag = 0.436 kg (5%)
gypsum = 0.139 kg (1.5%)

lime = 0.092 kg (1%)
water = 0.375 kg

SET F: set C + lime & gypsum #1
agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5%)
flyash = 0.218 kg (2.5 %)
gypsum = 0.057 kg (0.65%)
lime = 0.029 kg (0.33%)
water = 0.360 kg

SET G : set C+lime & gypsum #2
agg. = 5.8 kg
sand =2.8 kg
P.C. = 0.218 kg
flyash = 0.218 kg
gypsum = 0.087 kg (1%)
lime = 0.044 kg (0.5%)
water = 0.362 kg

SET H: set C + sodium silicate #1
age. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg
flyash = 0.218 kg
sodiumsilicate = 0.0218 kg (5% OF THE BINDING MATERIAL)
water = 0.357 kg

SET I: set C + sodium silicate#2
same as set H but with 0.0436 kg of the sodium silicate which is 10% of the total binding material in the mix.

RESULTS:

SET	COMPRESSIVE STRENGTH, MPa	
	28 DAY	56 DAYS
A	3.05	3.62
B	1.90	1.82
C	5.20	6.02
D	2.58	2.95
E	2.47	3.1
F	6.05	6.23
G	7.0	7.1
H	4.45	4.12
I	3.33	4.26

DISCUSSION :

- Although sets A, B and C were cast at the same day, meaning that the moisture content and the size distribution of sand and/or aggregate were exactly the same, the result for set C is suspicious. Since all the chemicals were added to sets B & C, the increase in strength in other mixes will be just compared to those two sets.

AFTER 56 DAYS OF CURING :

- Set D had a 62% higher U.C.S than the control set B.
- Set E indicated that increasing the amounts of lime and gypsum in set B caused a 8% increase in strength compared to set D, indicating that higher strengths could not be obtained with additional gypsum and lime in P.C./slag mixes. The additional gypsum and lime did act as void fillers in the samples and did not show any binding properties. Increasing the amounts of lime and gypsum by 50% caused only a 5% increase in strength.

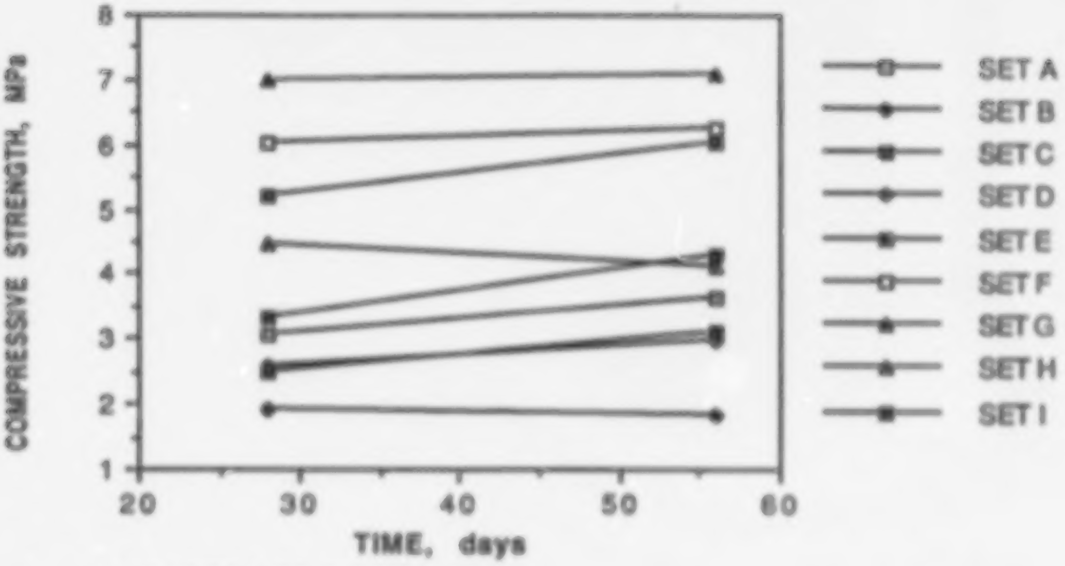
- Set F increased the U.C.S. by 3% compared to control set C. By increasing the amounts of lime and gypsum in the mix, set G, the U.C.S. was increased by 18% compared to control set C, which had a relatively high strength. This means still there might be a possibility of obtaining a higher strength with additional increase in lime and/or gypsum content in samples containing flyash.

- Addition of sodium silicate to the mix decreased the U.C.S of the control C mix. The strength was decreased by 32% in set H and 30% in set I. Both sets H & I indicated that the addition of sodium silicate would not have any positive effect on the strength development. In other applications which a much faster increase in strength is needed, the addition of this chemical might be beneficial.

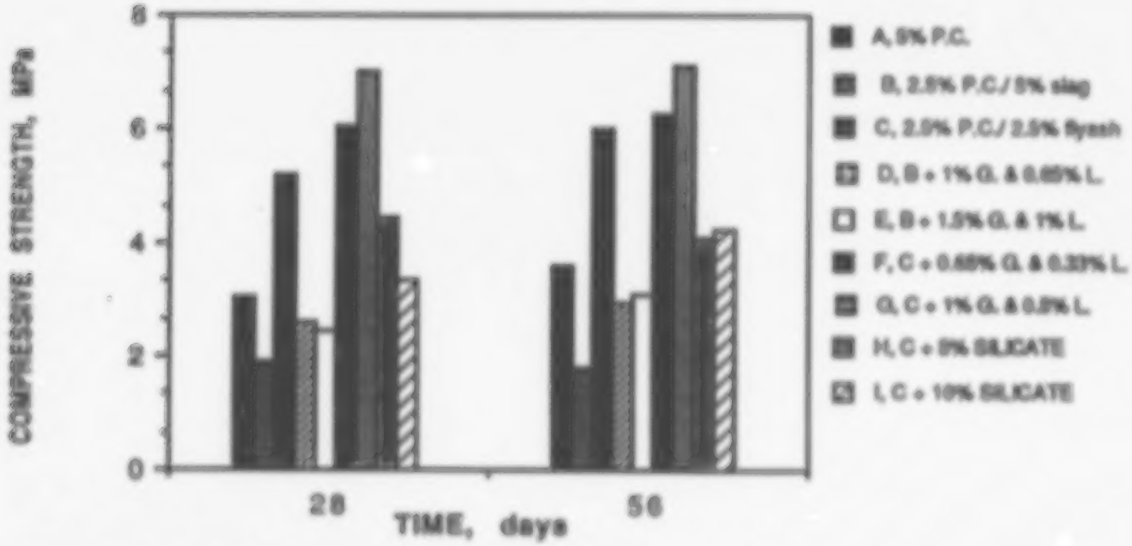
- It should be noticed that the rate of increase in compressive strength from 28 to 56 days is much higher for the control sets. This indicates that the addition of chemicals to the backfill would be beneficial in increasing the early strength of the fill material and most likely would not effect the strength of the fill in longterm.

- The rate of increase in compressive strength was much more pronounced in samples containing copper slag and an increase of up to 70% was obtained in set E, by comparing to control set B. The highest strength increase in samples containing flyash was obtained in set G by 17%, compared to set C. It should, however, be noted that the compressive strengths of the samples containing flyash were much higher compared to the ones with slag.

- Again it is reminded that most of the results were compared with set C, which is exceptionally high. Even using set C as the control set, the results indicated that the chemicals could have major effect in increasing the early strength of the backfill.



graph # 27 : RESULTS OF REPORT #9



graph #28 : RESULTS OF REPORT #9

REPORT # 10

ADDITION OF LIME, GYPSUM AND ANHYDRITE TO KCM BACKFILL (FINAL TEST RESULTS)

OBJECTIVE: To investigate the effect of addition of lime, gypsum and anhydrite to increase the early and/ or longterm compressive strength of P.C./flyash mixes. Addition of the above materials to P.C./copper slag binder was also investigated.

PROCEDURE :

All tests were carried out using blended Type 10 cement and Type C flyash. KCM copper slag of 3000 Blaine was used in these tests. Anhydrite, lime and gypsum were all pulverized to a size of approximately 2500 Blaine.

Each set was made of six 3-in. diam. cylinders to be tested at 28 and 56 day curing. The materials used in these sets were:

SET A: CONTROL, 100% P.C.
agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.435 kg
water = 0.357 kg

SET B: CONTROL (P.C. AND C FLYASH)
agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5 %)
flyash = 0.218 kg (2.5%)
water = 0.357 kg

SET C : CONTROL (P.C. AND SLAG)
agg. = 5.8 kg
sand = 2.8 kg
P.C. = 0.218 kg (2.5%)
slag = 0.436 kg (5 %)
water = 0.366 kg

SET D: set B+ 1% gypsum + 0.65 % lime
gypsum = 90 gr. (1%)
lime = 60 gr. (0.65 %)
water = 363 gr.

SET E : set B + 1% gypsum
gypsum = 90 gr. (1.0%)
water = 360 gr.

SET F: set B + 3% gypsum
gypsum = 270 gr., 3%
water = 368 gr.

SET G : set B+ 1% anhydrite + 0.65% lime
anhydrite = 90 gr.

lime = 59 gr.
water = 363 gr.

SET H: set B+ 1% anhydrite
anhydrite = 90 gr.
water = 360 gr.

SET I: set B + 3% anhydrite
anhydrite = 270 gr.
water = 368 gr.

SET J : set B + 5% anhydrite
anhydrite = 450 gr.
water = 375 gr.

SET K : set C + 1 % gypsum + 0.65% lime
gypsum = 92 gr.
lime = 60 gr.
water = 372 gr.

SET L : set C + 1% gypsum
gypsum = 92 gr.
water = 369 gr.

SET M: set C + 3% gypsum
gypsum = 276 gr.
water = 376 gr.

SET N : set C + 1% anhydrite + 0.65% lime
anhydrite = 92 gr.
lime = 60 gr.
water = 372 gr.

SET 0 : set C + 1% anhydrite
anhydrite = 92 gr.
water = 369 gr.

SET P : set C + 3% anhydrite
anhydrite = 276 gr.
water = 376 gr.

RESULTS:

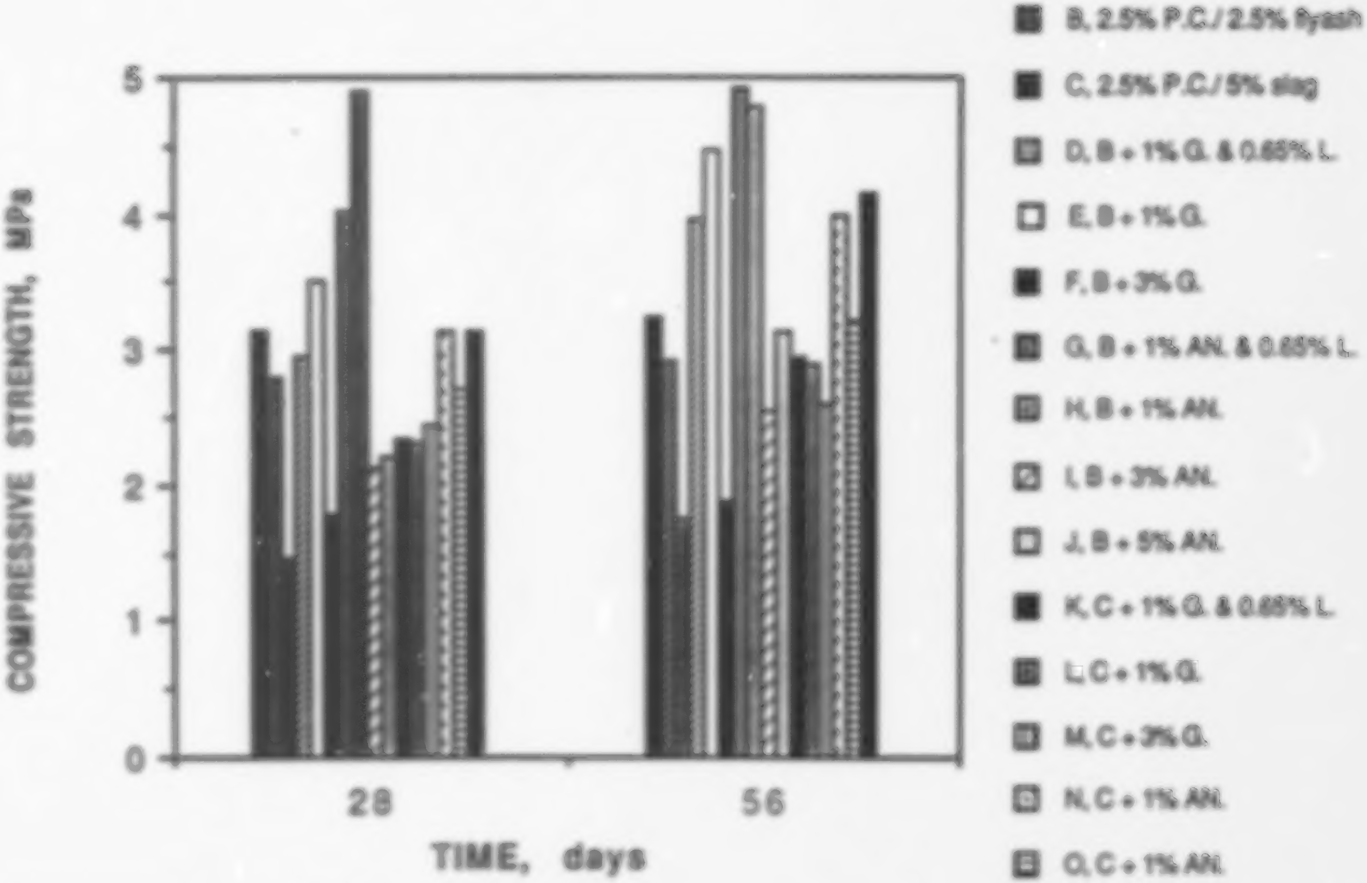
SET	COMPRESSIVE STRENGTH, MPa 28 DAY	56 DAYS
A	3.15	3.25
B	2.78	2.92
C	1.45	1.75
D	2.96	3.95
E	3.5	4.46
F	1.78	1.90
G	4.02	4.93
H	4.9	4.80
I	2.13	2.55

J	2.2	3.15
K	2.35	2.95
L	2.31	2.9
M	2.44	2.6
N	3.13	4.0
O	2.7	3.23
P	3.15	4.15

- Sets D, E, G, H, N and P, have all surpassed the strength development of control set A after 28 and 56 days of curing. These increased strengths were achieved even with partial replacement of the cement with Type C flyash or Kidd Creek Copper slag.
- The highest compressive strength was obtained with set G which had 38% and 52% higher strength compared to control set A after 28 and 56 days of curing respectively.
- The rate of strength development is almost constant for the control set A after 28 days of curing. With addition of gypsum, anhydrite and/or lime the samples have a equal or better early strength than set A and with having a longer curing cycle the longterm strength of the samples are also superior than control set A. The % rate of increase in strength was generally between 20 to 30% from 28 to 56 days of curing. This could result in even better strength development of the mixes after longer curing time.
- The only sample which had a constant strength from 28 to 56 days of curing was set H, which had the highest 28 day strength and the second highest 56 day strength .
- Comparing sets D and E showed that addition of lime has decreased the strength by 12% after 56 days of curing, although, it caused an increase of 18% for the 28 day strength.
- Comparing sets E and F showed that the addition of more than 1% gypsum to the mix decreased the strength by 58%. This could be due to lack of enough water for the gypsum to completelyreact.
- Sets G and H showed that addition of lime to the samples containing anhydrite decreased the 28 day strength by 18% and had increased the 56 day strength by just 3%. This indicated that the addition of lime to anhydrite samples had no major effect in strength development.
- Increasing the anhydrite content from 1 to 3 and 5% caused a decrease in compressive strength of the samples, by comparing sets H, I and J.
- Set C, with slag as a partial replacement of cement, had a low cementitious properties. However, the addition of anhydrite, gypsum and/or lime to control set C increased the strength in all mixes.
- The best result was obtained when using 3% anhydrite, set P, which increased the strength of the control set C by 217% and 240% after 28 and 56 days of curing respectively.
- Set N, with just 1% anhydrite and 0.65% lime had almost the same 28 and 56 day strengths as set P, with 3% anhydrite.
- All the slag/P.C. samples with addition of anhydrite had equal or better 28 and 56 day strengths compared to set A.

DISCUSSION:

The results from these tests were encouraging and small addition of anhydrite and/or gypsum significantly increased the strength of the backfill mix. The addition of these materials also increase the early strength of the samples containing slag or flyash replacement for P.C. This would not limit us to a specific mining cycle time when these material are used. Using copper slag with addition of anhydrite, gypsum and /or lime as a partial replacement of P.C. would make a very attractive backfill mix. More comprehensive tests on the exact behaviour of anhydrite, gypsum and their optimum content in the mix should be carried out.



graph #29 : RESULTS OF REPORT #10

REPORT # 11

SLURRY DISPERSANT TEST RESULTS, Test #1

OBJECTIVE : To study the effect of slurry dispersant on the compressive strength development of backfill mix. Based on the previous test results of Grace's 'Hydrafill' (slurry dispersants), these series of test cylinders were cast using a lower water to cement ratio. Observations of the previous test cylinders indicated that the W/C ratio used may have been too high. It appeared that the slurry washed through the aggregate, settling at the bottom of the cylinders, and leaving a section of aggregate with little or no slurry coating. It was anticipated that with the correct W/C ratio, dispersion of the cement slurry would be more uniform thus resulting in a higher accuracy in compressive test results.

PROCEDURE :

A total of 45, 6-inch diameter, test cylinders were cast. A 60% Type C flyash (F.A.)/40% P.C. binder mix was used. The flyash was supplied by Canada Cement Lafarge. A ratio of 280 ml Hydrafill per 100 kg (220 lb) cement binder was recommended for the test. Testing of the cylinders was performed at 14, 28 and 56 day cures. Feldman Lake potable water and a blended mix of 3 brands of P.C. was used. The water/P.C. ratio was 0.82 (Pulp density = 55%). The materials used in these sets were:

A: 60% flyash/40% Portland Cement binder
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	5.4
flyash	8.1
water	11.1

B: 60% flyash/40% P.C. + Hydrafill dispersant
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	5.4
flyash	8.1
water	11.1
Hydrafill	0.04

C: 60% flyash/40% P.C. + Hydrafill dispersant
minus 5% binder
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	5.1
flyash	7.7
water	10.5 1
Hydrafill	0.036

D: 60% flyash/40% P.C. + Hydrafill dispersant
minus 7.5% binder
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	5.0
flyash	7.5
water	10.2
Hydrafill	0.035

E: 60% flyash/40% P.C. + Hydrafill dispersant
minus 10% binder
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	4.9
flyash	7.3
water	10.0
Hydrafill	0.034

RESULTS :

		Compressive Strength (MPa)		
SET		14 Days	28 Days	56 Days
A:	60% F.A./40% P.C.	4.6	5.1	5.8
B:	60% F.A./40% P.C. + Hydrafill	9.1	9.9	11.6
C:	60% F.A./40% P.C. + Hydrafill - 5% binder	7.1	9.3	10.14
D:	60% F.A./40% P.C. + Hydrafill - 7.5% binder	6.6	8.1	10.12
E:	60% F.A./40% P.C. + Hydrafill - 10% binder	6.1	7.8	9.4

TABLE 1 : Compressive strength test results.

SET	Compressive Strength (MPa) Comparison to Controls are in **bold**					
	14 Days		28 Days		56 Days	
A: 60% F.A./40% P.C. (Control)	4.6	**100%**	5.1	**100%**	5.8	**100%**
B: 60% F.A./40% P.C. + Hydrafill	9.1	**198%**	9.9	**194%**	11.6	**200%**
C: 60% F.A./40% P.C. + Hydrafill - 5% binder	7.1	**154%**	9.3	**182%**	10.14	**175%**
D: 60% F.A./40% P.C. + Hydrafill - 7.5% binder	6.6	**143%**	8.1	**156%**	10.12	**174%**
E: 60% F.A./40% P.C. + Hydrafill - 10% binder	6.1	**133%**	7.8	**153%**	9.4	**162%**

TABLE 2 : Increase in compressive strength measured as percentage between Controls and different sets.

SET	Compressive Strength (MPa) 14 Days	 56 Days	 % increase
A: 60% F.A./40% P.C. **(Control)**	4.6	**5.8***	26%
B: 60% F.A./40% P.C. + Hydrafill	9.1	11.6	27%
C: 60% F.A./40% P.C. + Hydrafill - 5% binder	7.1	10.14	43%
D: 60% F.A./40% P.C. + Hydrafill - 7.5% binder	6.6	10.12	53%
E: 60% F.A./40% P.C. + Hydrafill - 10% binder	**6.1***	9.4	54%

* The 56 day Control cylinders were 5% lower in compressive strength than the 14 day Hydrafill -10% binder cylinders.

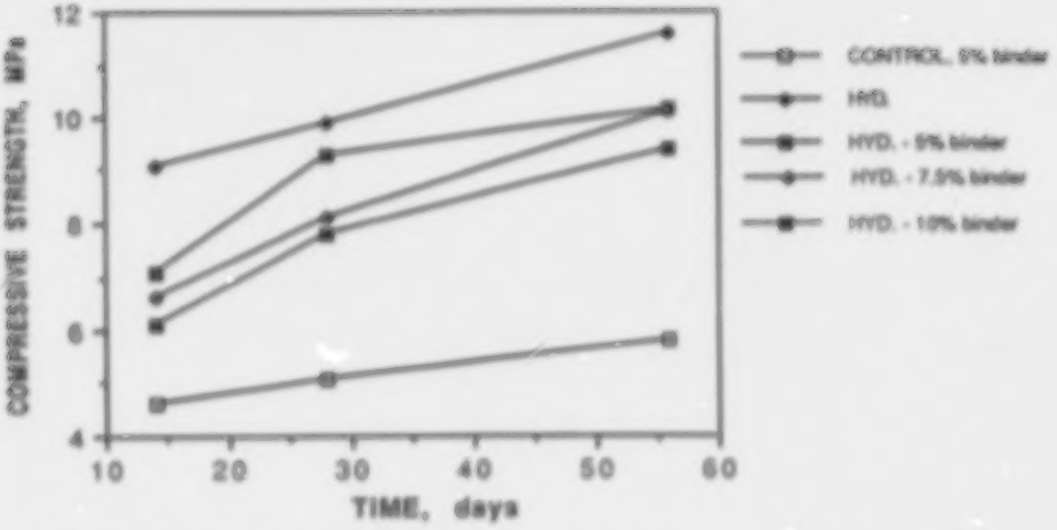
TABLE 3 : Percent strength increase between 14 days curing and 56 days curing.

DISCUSSION :

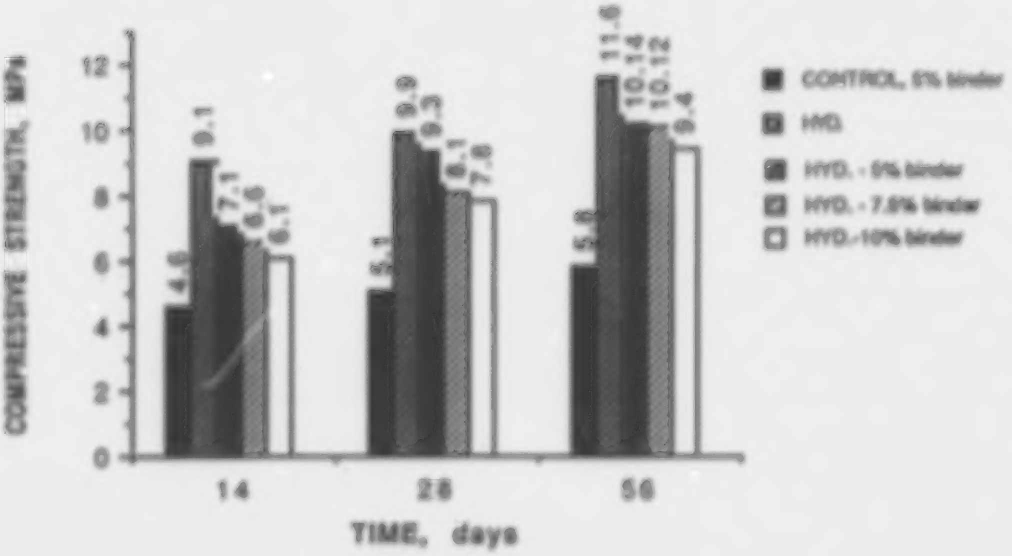
Tests showed that the strengths of the Hydrafill specimens were increasing steady even after 56 days of curing, while the control specimens showed a small increase rate after 28 days. More testing is required to arrive at the optimum reduction in binder while obtaining acceptable backfill compressive strengths. Operational aspects in employing slurry dispersant, such as possible foaming problem in the mixing tanks, should also be investigated.

CONCLUSION :

The use of slurry dispersant such as Hydrafill could have a positive effect in prolonging the curing process of the backfill mix, hence, allowing backfill mix to obtain a higher compressive strength.



graph #30 : RESULTS OF REPORT #11



graph #31 : RESULTS OF REPORT # 11

REPORT # 12

ANTHES VS GRACE SLURRY DISPERSANT, Test #2

OBJECTIVE : To study the effect of two commercially available dispersants on the compressive strength of the fill mix. This study was to determine if slurry dispersants could be used in the backfill system to allow for a reduction in overall binder material resulting in cost savings.

PROCEDURE :

A total of 45, 6-inch diameter, test cylinders were cast. A 60% Type C flyash (F.A.)/40% P.C. binder mix was used. The flyash was supplied by St. Mary Cement Co. A ratio of 0.5 kg Anthes "S-613" Superplasticizer per 100 kg cement binder was recommended for the Anthes test cylinders, and a ratio of 0.28 kg of W.R. Grace's "Hydrafill" dispersant per 100 kg cement binder was recommended for the Hydrafill test cylinders. Testing of the cylinders was performed at 14, 28 and 56 day cures. Feldman lake water and a blended mix of 3 brands of Portland cement was used. The water/cement ratio was 0.82 (Pulp density = 55%). The materials used in these sets were :

SET A : 60% flyash/40% Portland Cement binder.

(5% binder content)
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	5.4
flyash	8.1
water	11.1

SET B : 60% flyash/40% Portland Cement + Anthes less 10% binder.

(4.5% binder content)
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	4.9
flyash	7.3
water	10.0
Anthes	0.061

SET C : 60% flyash/40% Portland Cement + Hydrafill less 10% binder.

(4.5% binder content)
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	4.9
flyash	7.3
water	10.0
Hydrafill	0.034

SET D : 60% flyash/40% Portland Cement + Anthes less 20% binder.

(4.0% binder content)

(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	4.3
flyash	6.5
water	8.9
Anthes	0.054

SET E : 60% flyash/40% Portland Cement + Hydrafill less 20% binder.
(4.0% binder content)
(mix for 9 cylinders, weight in lbs)

aggregate	180.0
sand	90.0
P.C.	4.3
flyash	6.5
water	8.9
Hydrafill	0.030

RESULTS :

SET	Compressive Strength (MPa)		
	14 Days	28 Days	56 Days
A: 60% F.A./40% P.C.	2.5	3.4	4.3
B: 60% F.A./40% P.C. + Ant. -10% binder	2.4	3.3	5.0
C: 60% F.A./40% P.C. + Hyd. -10% binder	rejected	3.0	3.9
D: 60% F.A./40% P.C. + Ant. -20% binder	1.4	2.0	3.5
E: 60% F.A./40% P.C. + Hyd. -20% binder	1.3	2.2	3.7

OBSERVATIONS

- The dispersant and superplasticizer test cylinders having 10% less binder had compressive strength results comparable to or better than the Control cylinder results.

- The Anthes "S-613" superplasticizer and W.R. Grace's "Hydrafill" had comparable results throughout the three cures when using the recommended dosages. The dosages recommended for each of the products are as follows;

- Some settling of fines was still noticed in all cylinders that contained superplasticizer or dispersant, but settling was very little in control cylinders.

- Three of the 18 Hydrafill cylinders broke prematurely while being removed from their plastics molds. The Hydrafill cylinders were still wet at the 14 day test, but were properly cured at the 28 day test.

- The results of series #2 were reasonable and acceptable, report #12. Although the same binder material, aggregates, sand, chemicals, water/cement ratio, water, and procedures were used in this series and in series #1, report #11, the compressive results of the controls differ considerably from one another.

	14 day	28 day	56 day
- Control cylinders of series#1 Slurry dispersant test; MPa,	4.6	5.1	5.8
- Control cylinders of series#2 Slurry dispersant test;	2.5	3.4	4.3

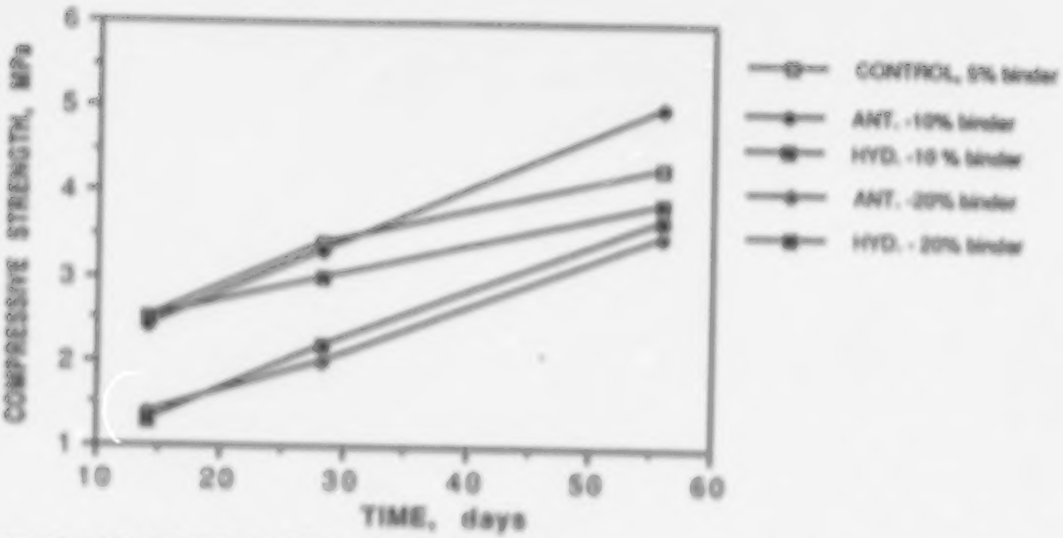
- The variation in test results between series #1 and #2 was possibly caused by the different temperature of the environment in which the cylinders were cast. This factor may have hampered the normal curing of the test cylinders resulting in different compressive strength results from one series to another. Although every attempt was made to duplicate this series exactly as it was performed previously, more care should be taken for future tests especially in the area of material and curing temperatures.

DISCUSSION :

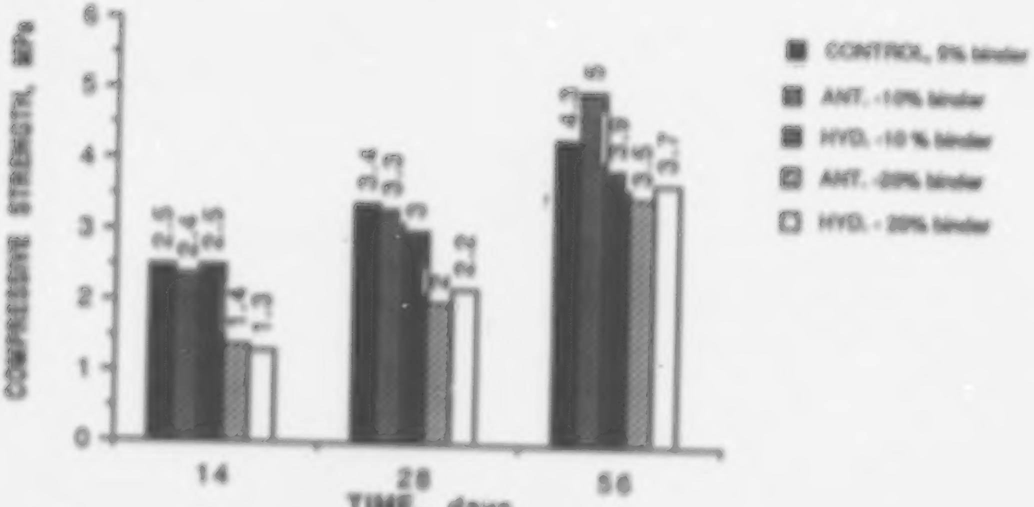
Considering that the results of the test cylinders using 10% less binder are comparable in compressive strength to the control cylinders, a large scale test using 5% less binder should be attempted. In future testing, more attempts will be made to duplicate material temperatures and conditions to allow comparison from one series of test results to another.

CONCLUSION :

Results indicated, by adding dispersants, test cylinders having 10% reduced binder material had a comparable strength to the control cylinders which had no reduction in binder material. Further testing should be considered to determine optimum slurry dispersant dosages and to study the effect of excess water (higher W/C ratio) on the dispersant.



graph #32 : RESULTS OF REPORT #12



graph #33 : RESULTS OF REPORT #12

REPORT # 13

TEKPAK MATERIAL FOR STOPE CAPPING, Test #1

OBJECTIVE: To study the possibility of using monolithic materials such as Tekpak, a product of Technik Mining, as a means of capping the top of the stope for tight sand filling.

PROCEDURE:

- Hand mixing was adapted in 20 gallon containers. Tekcem and Tekbent were mixed separately in 6"- diam. cylinders before being blended together in a 20 gallon container.
- Three sets of Tekpak pours were tested:

A. In the first test, two 6"-diam. cylinders were filled with Tekcem and Tekbent separately to 1/3 of their 12" height. The remaining volumes were filled with underground water. After mixing Tekcem and Tekbent together, the final mix was allowed to set for five minutes before pouring.

B. In the second test, 1/2 of cylinders were filled with Tekcem and Tekbent materials separately and then the remaining volumes were filled with water. The set time for final mix was again 5 minutes.

C. The third test was the same as B with 3 minutes set time.

OBSERVATIONS :

- The penetration into the fill was about 4 cm for set A, 2.5 cm for set B and 1 cm for set C, after three minutes setting time . Average penetration distance for all sets was around 4 cm.
- Tekcem is much denser than Tekbent and most likely more than the standard amount of Tekcem was used in our testing.
- Cracks were noticed on set C, 3 minutes setting time, and some water went through the Tekpak capping material.

DISCUSSION:

- This underground trial showed that the Tekpak material was a suitable water sealant for stope capping. If Tekpak is mixed properly and the setting time for the final mix is adequate then there should be no cracks on the surface of the material for water to go through. The compressive strength of the Tekpak in 2 days should be around 3-4 MPa and this would enable it to handle the weight of the sand slurry on top. More definite results will be obtained with lab tests and future underground trials. The most important disadvantages of Tekpak is that cold water and low ambient temperatures would considerably decrease the early strength of the material. The sensitivity of Tekpak to above factors will be obtained with future testings.

CONCLUSION :

- Tekpak looked solid after 24 hours of curing, therefore, it should be able to handle the weight of the sand slurry on top. Water was poured on the 24 hour old Tekpak samples and they showed good water sealant characteristics. Water did not penetrate at all and it ran down the slope of the muckpile.

RECOMMENDATIONS :

- We had problem with the type of mixer that was used and large amounts of unmixed material remained at the bottom of the mixing tank, especially for Tekcem mix. A more suitable mixer will be required for future work.
- If possible, these tests should be carried out on 2 or 3 main levels underground to see the effects of water and ambient temperatures on curing of Tekpak.
- Other alternatives such as Flashpak (flyash, P.C. and accerelator) should also be tested underground.

FUTURE WORKS:

- Lab tests on the effect of the water quality on compressive strength of the Tekpak is ongoing. Comparison will be made between recycled water and the underground water. The last set of lab results were ignored due to the high heat dissipation property of the brass moulds used for testing.
- Lab tests should be carried out to study the sensitivity of the Tekpak to different water and possibly ambient temperatures.

REPORT # 14

TEKPAK MATERIAL FOR TIGHT FILLING, Test #2

OBJECTIVE : To study the possibility of using monolithic materials such as Tekpak as a means of capping the top of the stope for tight sand filling. This was to repeat the previous test work, Report #13, with a modified mixer.

MODIFICATIONS:

- Certain modifications had to be made to the mixer to improve its performance. The usual double-drum mixer used for diamond drilling which was used in first underground trial is suited for mortar-like slurries. This mixer did not agitate the materials enough, especially Tekcem, to achieve a complete chemical reaction between the two components. The modifications to the mixer were as follows:
- Four 3-in wide baffles were attached vertically to the inside wall of each drum. This will help the mixer to achieve the required turbulence and breaking up the rotation of the liquid body.
- Rubber sweepers were added to the mixer blades to get it as close as possible to the floor of the mixing drums.

MATERIALS :

- 15 - 25 kg bags of Tekcem and Tekbent each.
- 1 modified double drum mixer.
- 1 sand piper pump.
- Three 1-in air hoses and one 1-in water hose.
- One 5 gal. pail for measuring the exact amount of water required.

PROCEDURE:

Measured amounts of water were poured in each drum. A 25 kg bag of Tekcem were added to the lower drum and a bag of Tekbent to the upper drum. The material were then mixed individually for the specified amount of time which is shown in Table 1. Tekbent from the upper drum was then poured into the Tekcem mix by opening the outside valve on the drum. The Tekpak was then mixed for the required time and pumped and sprayed on top of the muckpile using a spidel pump. Two 3-in diam. cylinder samples were taken for each set just before pumping the Tekpak. The poured area was then outlined and numbered for future observations.

OBSERVATIONS :

- In sets 1-6, set #2 looked the best and had better capping characteristic than other sets.
- Sets 4 and 10 looked very watery and had bubbles on the surface of the final mixes which were to be sprayed. This was due to short blending time.
- Set 12 had the best spraying characteristics and the slurry was the thickest.

CYLINDER SAMPLES :

- 3-inch diameter cylinder samples were collected from sets 7 to 12. No cylinder samples were taken for sets 1-6.
- Sets 7-12 were all set within 30 minutes of blending the Tekpak, except set #10. Set #10

had about 5 cm of water on top of the cylinders. This was most likely due to inadequate blending time.

- Set #7 had the most surface cracks and the second worst was set #8.
- Sets 9, 11 and 12 looked solid with no cracks on their surfaces.
- The 3-day compressive strength for all samples were low and set #11 had the highest compressive strength, about 0.6 MPa. The low strength of these cylinders could have been due to inability to strip the cylinder moulds right after the initial setting time. The moulds were stripped from the cylinders just before the compressive strength tests were carried out.

DISCUSSION:

Tekpak did set immediately after blending the two components and was solid in a very short time. The short setting time of these materials could find other applications underground where pre-made forms contain the final mix. This material would have easily capped the top of the muckpile if the final blended mix could have had standing time before pumping. The standing time was not practical because of the thickness of the slurry and the difficulty in pumping. In first underground trial all sets that were tested showed excellent capping characteristics and that was due to the allowed standing time before pouring.

- Sets 1, 3 and 10 were watery and this was caused by inadequate blending time.
- Sets 9, 11 and 12 resulted in a much thicker slurry, especially set # 12, and these sets also had better spraying characteristics.
- Set # 13 could not be completely pumped due to the thickness of the slurry . The two components start to set immediately after mixing and became difficult to pump.
- The compressive strength obtained from 3 day cured 3-in diam. cylinders were significantly lower than the actual strength obtained in laboratory tests.
- The clear coating on the fill material was not visible and this was most likely due to high pressure spraying.
- Blending time plays a more important role for strength development characteristics of Tekpak than individual mixing time.

CONCLUSION :

This underground trial indicated that the Tekpak material could not act as stope capping agents if high pressure spraying or long vertical falls are employed. This material showed a good sealing characteristics when slow rate of pouring was used. This material is as costly as concrete at the site underground, while it needs more material for most jobs due to its lower strength.

FUTURE WORK :

- Underground testing should be repeated with the following considerations:

1 - To lower the speed of pumping the slurry, amount of the slurry used in each batch should be cut in half. This will allow for a much slower pumping rate before the Tekpak starts to set. This means that half of the bag of Tekcem and Tekbent should be used for each set.

2 - Another alternative could be pouring the slurry on top of the muckpile using a 5 gallon bucket. This method of pouring was tried in the first underground trial and the muckpile was easily capped by the slurry.

3 - Carry out same tests on different sections of the muckpile, fine and coarse sections, to find the amount of Tekpak needed for capping these sections.

SET #	STANDING TIME, MIN	BLENDING TIME	WATER/TEK RATIO
1	3	3	3

2	3	5	3
3	3	7	3
4	5	3	3
5	5	5	3
6	5	7	3
7	3	3	2.5
8	3	5	2.5
9	3	7	2.5
10	5	3	2.5
11	5	5	2.5
12	5	7	2.5
13	3	5	3

TABLE 1 : MIXING PARAMETERS OF ALL DIFFERENT SETS

REPORT # 15

IMPROVING EARLY STRENGTH AND SETTING TIME OF KCM SANDFILL

OBJECTIVE: To study the effect of addition of sodium silicate and two other commercially available chemicals to improve the early strength and setting characteristics of the present KCM sandfill. This would allow a controlled setting time for the sandfill slurry, thereby making tightfilling the stopes possible.

PROCEDURE:
All tests were carried out using blended Type 10 P.C. and Type C flyash. Each set was made of six 3-in. diam. cylinders to be tested at 28 and 56 day time intervals. The chemicals used for testing were sodium silicate, Hydroset, a product of W.R. Grace Chemical and Tekfill of Technik Mining. Sand and water (with specified dosage of the chemicals) were mixed together in a mixer for a period of 2 minutes and at the same time the P.C. /flyash slurry was blended together in a separate mixer. The P.C./flyash slurry of 60% pulp density was added to the sand slurry of 68% pulp density and they were mixed together for 4 minutes. The final mix was then poured into 3-in diam. cardboard cylinders and allowed to cure. Because of a the low strengths of some sets the samples were demolded after 28 days of curing.

SET A: CONTROL, 100% P.C.
sand = 4800 gr. (all weights in gr.)
P.C. = 480
water = 2577.6 (2256 gr with sand slurry and 321.6 gr. with cement slurry)
SET B: CONTROL, 50% P.C. /50% FLYASH
sand = 4800 gr.
P.C. = 240
flyash = 240
water = 2577.6
SET C : CONTROL B + 1% SODIUM SILICATE
sand = 4800 gr.
P.C. = 240
flyash = 240
sodium silicate = 78.6 (was mixed in sand slurry)
water = 2577.6

SET D: CONTROL B + 2% SODIUM SILICATE
same as set C but with 157.2 gr. of sodium silicate.
SET E: same as set C but replacing the sodium silicate with Hydroset of Grace Chemicals.
Hydroset= 78.6 gr.
SET F: same as set D but replacing the sodium silicate with 2% Hydroset.
Hydroset = 157.2 gr.
SET G : same as set C but replacing the sodium silicate with Tekfill of Technik Mining Co.
Tekfill =78.6 gr.
SET H: same as set D but replacing the sodium silicate with Tekfill.
Tekfill = 157.2 gr.

All the chemicals were in liquid form. These chemicals when added to hydraulic fill did react rapidly with the water and the binding material to accelerate solidification with minimum loss of water.

RESULTS :

SET	A	B	C	D	E	F	G	H
28 days, U.C.S, MPa	0.7	0.71	1.05	0	0	0	1.2	0
56 days, U.C.S, MPa	0.87	0.87	1.5	0	0	0	1.56	0.43

OBSERVATIONS:

- Just after pouring the samples the amount of bleeding water which was siting on the top of the samples was measured and it was as follow:
set A= 4 cm, set B= 3.3, set C=2, set D= 2, set E= 3.3, set F= 3, set G= 3 and set H= 0.5 cm.
- After 1 day of curing control set A looked real solid and had hardly any water left on the top of the samples.
- Set D, E and F were still very soft and the same amount of bleeding water was still on top of the samples.
- Set G looked even better than the control set A.
- Both sets C and G looked very light in colour compared to the rest of the samples.

RESULTS : (after 56 days of curing)

- Sets C, D & H were not completely dry and could have higher longterm strength .
- Set C did not show as much residual strength as it did for 28 day samples.
- Set D, E and F had very low compressive strengths and sets E and F could not be taken out of the moulds without being broken .
- Set C had the highest strength increase rate comparing 28 and 56 day test results. The increase in strength of set C after extra 28 days of curing was around 40%. Set C had almost the same strength as set G after 56 days of curing. The percentage rate of increase in set G was 30%, second highest.
- The percentage rate of strength increase for sets A & B was 24%. This showed that the replacement of P.C. with flyash did not decrease the strength of the sandfill samples.

DISCUSSION :

- Hydroset from Grace Chemicals did not perform satisfactory. The samples made with Hydroset had a low compressive strength after 56 days of cure.

- Addition of 1% sodium silicate or Tekfill to the present sandfill slurry could increase the compressive strength by almost 77% after 56 days of curing. By increasing the dosage to 2% of the total material, the strength of the samples at 28 and 56 day testing intervals were zero. This indicated that the optimum amount of sodium silicate in the mix should be much lower than 2%. More work in the optimum dosage of the sodium silicate should be carried out.

- Tekfill performed well and increased the compressive strength of the control set B by almost 79% after 56 days of curing when 1% Tekfill was added. The 2% dosage of the Tekfill to the sandfill slurry performed exactly the same as Sodium Silicate in set D. and had a low strength of 0.43 MPa after 56 days of curing.

- A problem which might rise is the sensitivity oft he sandfill slurry to the dosage of the chemicals used and an adequate quality control is needed. The price of Tekfill is around 20% cheaper than type N sodium silicate which was used in this testing program. Price for a ton of Tekfill is approximately $280 whereas sodium silicate costs about $340/tonne.

CONCLUSION:

Based on the preliminary test results, it appears that both 1% Tekfill and sodium silicate additions to KCM sandfill could be beneficial in obtaining higher strength sandfill. The major problem with KCM sandfill is the run of slurry through the voids in CRF. More tests should be carried out to obtain a slurry which could possess both high strength and fast gelling characteristic for stope capping. It was found out in the later stages of testing that for investigating the gelling characteristics of the chemicals, such as Tekfill, they should be added in the final stage of mixing, just before the sandfill is placed in the stope.

REPORT # 16

SUBJECT: WATER RETENTION TESTS ON KCM SANDFILL USING CHEMICALS

OBJECTIVE : To investigate the water retention properties of KCM sandfill and possible use of chemicals to increase the amount of water retained in the sandfill slurry.

PROCEDURE:

This test was done by filling 12" high x 6" diameter cylinders. The bottom of each cylinder had 30 holes of 0.3 cm diameter to allow for drainage. The cylinders were weighed after 15 hours at the time when no more drainage in mixes was anticipated. Five different mixes were tested and the emphases was on the sandfill which contained 100% Portland cement as the binding agent. Since the chemicals which were being tested were known to be sensitive to water quality, underground water was used in all the mixes. Sand, water and/or required amount of chemicals were mixed for four minutes in the food mixer using the intermediate speed and a pulp density of 68%. At the same time the hand mixer was used to obtain a cement slurry with the pulp density of 60% which had a mixing time of 2 minutes. Both slurries were then mixed together for another 3 minutes in the food mixer. The final mix was then left alone for 40 seconds before pouring it into the appropriate cylinder.

The amount of material used in each mix was just enough to fill the 12" long cylinders to the top. The punctured cylinder was placed and taped on top of a solid cylinder for measuring the amount of material loss.

The materials of each mix were as follow :

MIX A: (CONTROL MIX, 100% P.C.)
sand : 6400 gr.
P.C. : 640 (10%)
water : 3436 (3008 gr. with sand slurry & 428.8 gr. with cement slurry for the required pulp density)

MIX B: CONTROL A + 1% SODIUM SILICATE
sodium silicate = 104.7 gr.

MIX C : CONTROL A + 2% SODIUM SILICATE
sodium silicate = 209.6 gr.

MIX D : CONTROL A + 1% TEKFILL

MIX E : CONTROL A + 2% TEKFILL

OBSERVATIONS:

- Mix A lost around half of its volume in the first 5-8 minutes after pouring . It started to stabilise after 15 minutes and by then it had lost 2/3 of its volume. There was around 5 cm of bleed water on top of the cylinder which was drained completely after one hour through a weak zone in one side of the cylinder. The low strength of the control mix did not allow it to sustain the bleed water more than one hour.

- Mix B was drained much slower compared to mix A and it stabilised after losing 1/3 of its volume which took around 15 minutes. 2 cm of bleed water was trapped on top of the cylinder, 45 minutes after pouring.
- Mix C looked very much like mix B except it had 2.5 cm of bleed water after 35 minutes. The loss of volume in this mix was exactly the same as mix B, meaning that increasing the amount of sodium silicate did not increase the flow characteristics.
- Mix D had lost around 1/4 of its volume after stabilising in about 20 minutes. It had around 8 cm of bleed water on top of the cylinder.
- Mix E looked the same as mix D with only 1 cm of bleed water.

15 hours after pouring the mixes :

- There was no evidence of bleed water on top of any of the samples.
- By pressing the samples the strength of the samples were rated in the order as follow:
HARDEST. MIX E, MIX C, MIX A, MIX D, MIX B. **SOFTEST**

RESULTS:

- The size of the hole which was used for drainage purposes, 0.3 cm, should have been smaller to allow minimum amount of cement and sand escape through it. This hole size was chosen for comparison purposes, since Technik Mining was doing the same tests using same size drain hole. The only difference in these two testing programs was that the chemicals were added at the initial stage at KCM, but at the final stage in the tests done by Technik Mining.

- In the bottom cylinders large amounts of solid were deposited and the weights of the solids which passed through the holes are listed below. In the bottom cylinders the complete separation of water and solids had taken place and the amounts of water on top of each mix is also presented.

MIX	TOP CYLINDER, kg	BOTTOMCYLINDER, kg	WATER, kg
A	3.6	5.1	1.33
B	4.9	3.3	0.98
C	4.8	3.4	0.97
D	5.8	2.8	1.02
E	5.1	3.3	1.06

- The weight of an empty cylinder was around 0.22 kg which is included in the top and bottom cylinder readings.

- The increase in water retention of each mix compared to control mix A is as follow mix A= 100%, mix B= 145%, mix C= 135%, mix D= 162%, mix E = 142%, hence, mix D performed the best.

- If the loss of sandfill material (water and solids) in the control mix A is assumed to be 100% the remaining percentages are as follow:
mix B= 64%, mix C= 66%, mix D= 55%, and mix E= 64%.
- If the amount of pour water in the bottom cylinder of the control A mix is assumed to be 100%, water loose in other mixes are as follow:

mix A= 100%, mix B= 61%, mix C= 59%, mix D= 68%, mix E=61% . Mixes B, C & E perform almost the same.

- The total material retained in the top of each cylinder compared to the original amount of material is as follows:
mix A = 40%, mix B= 61%, mix C= 59%, mix D= 68%, and mix E= 61%.

APPENDIX B

RESULTS OF IN SITU STRESS METER INSTRUMENTATION
FOR
KIDD CREEK [illegible]CKFILL STOPES

Falconbridge Ltd. / 1990

Layout of Irads and extensometers:

NAME	AZ	DIP	LENGTH	SIZE	ANCHORS
G13	45	-25	13	AX	5,10,12 m
I-24	315	+45	10	EX	
I-25	135	+45	10	EX	
G-14	45	-25	14	AX	5, 8, 13 m

G-15 was not installed due to low RQD of fill.

-STRESSMETER CALIBRATION:

To relate stressmeter reading to a value for rock stress, following equation was used.

$$\sigma_r = \frac{((422400)^2 * (1-(To/T)^2)}{9.4 - 0.5 * 10^{-6} * Er}$$

where:

σ_r = stress change

TO= initial meter reading

T = later meter reading

Er = Young's modulus of the rock = assumed , $11.5*10^6$

which gives a value of change in the uniaxial stress in a block of rock.

631 STOPE BLASTING

SLOT BLASTS:

38-1 SLOT FINAL - MAY 20 - DAY 140
38-2 SLOT FINAL - JUNE 16 - DAY 166

U/C BLASTS:

U/C STARTED - APRIL 23 - DAY 113
U/C FINISHED - APRIL 27 - DAY 117

RING BLASTS:

38-1 RING 1 TOES - MAY 14, DAY 134
38-1 RING 1 FINAL - MAY 22, DAY 142
38-1 RINGS FINAL - JUNE 5, DAY 156

38-2 RINGS SOUTH OF SLOT, 1,2 & 3 - JUNE 19, DAY 170
38-2 FINAL, JULY 2, DAY 183

INST. No.	I - 24				
		T	STRESS	TOT. STR.	
DAYS	last read	cur. read	CHANGE	CHANGE, psi	REMARKS
28	2984	2979	-18.4438		To = 2984
35	2979	2977	-7.4036	-25.8474	
42	2977	2976	-3.7074	-29.5548	
47	2976	2975	-3.7111	-33.2659	
49	2975	2973	-7.4335	-40.6994	
54	2973	2973	0.0000	-40.6994	
56	2973	2974	3.7186	-36.9808	
63	2974	2969	-18.6307	-55.6114	
68	2969	2970	3.7337	-51.8778	
70	2970	2963	-26.2151	-78.0928	
75	2963	2969	22.4814	-55.6114	
84	2969	2967	-7.4787	-63.0901	
89	2967	2966	-3.7450	-66.8351	
91	2966	2966	0.0000	-66.8351	
96	2966	2966	0.0000	-66.8351	
104	2966	2967	3.7450	-63.0901	
112	2967	2967	0.0000	-63.0901	
117	2967	2971	14.9422	-48.1479	
118	2971	2956	-56.3465	-104.4944	
119	2956	2957	3.7831	-100.7112	
119	2957	2958	3.7793	-96.9320	
120	2958	2954	-15.1402	-112.0722	
126	2954	2960	22.6873	-89.3849	
132	2960	2958	-7.5471	-96.9320	
139	2958	2967	33.8419	-63.0901	
146	2967	2966	-3.7450	-66.8351	
152	2966	2981	55.7799	-11.0552	
153	2981	2980	-3.6925	-14.7476	
154	2980	2980	0.0000	-14.7476	
159	2980	2995	54.9994	40.2518	
161	2995	2995	0.0000	40.2518	
166	2995	2934	-228.9562	-188.7044	
168	2934	2895	-154.0268	-342.7312	
173	2895	2917	87.6462	-255.0850	
175	2917	2914	-11.8350	-266.9199	
180	2914	2916	7.8940	-259.0259	
182	2916	2915	-3.9450	-262.9709	
187	2915	2957	162.2596	-100.7112	
189	2957	2954	-11.3609	-112.0722	
196	2954	2950	-15.2018	-127.2740	
201	2950	2947	-11.4420	-138.7160	
203	2947	2945	-7.6475	-146.3635	
208	2945	2940	-19.1870	-165.5505	
210	2940	2930	-38.6690	-204.2195	
215	2930	2930	0.0000	-204.2195	
217	2930	2911	-74.5721	-278.7915	
221	2911	2924	51.1800	-227.6116	
223	2924	2929	19.5034	-208.1082	
228	2929	2923	-23.4161	-231.5243	
231	2923	2923	0.0000	-231.5243	
238	2923	2924	3.9127	-227.6116	
269	2924	2926	7.8134	-219.7982	
275	2926	2949	88.7141	-131.0841	
288	2949	2951	7.6164	-123.4678	
295	2951	2952	3.8024	-119.6654	
303	2952	2951	-3.8024	-123.4678	
309	2951	2949	-7.6164	-131.0841	
314	2949	2949	0.0000	-131.0841	
	2949	2951	7.6164	-123.4678	

TABLE 1 : measured stress change by I-24

INST.NO	..........I-25			
To= 2268				
	LAST	CURRENT	STRESS	TOTAL STRESS
DAYS	READING	READING	CHANGE, PSI	CHANGE, psi
28				
35	2268	2268	0.0000	
42	2268	2270	16.7383	16.7383
47	2270	2268	-16.7383	0.0000
49	2268	2267	-8.3858	-8.3858
54	2267	2267	0.0000	-8.3858
56	2267	2266	-8.3969	-16.7827
63	2266	2264	-16.8271	-33.6098
68	2264	2264	0.0000	-33.6098
70	2264	2264	0.0000	-33.6098
75	2264	2264	0.0000	-33.6098
84	2264	2264	0.0000	-33.6098
89	2264	2265	8.4191	-25.1907
91	2265	2264	-8.4191	-33.6098
96	2264	2264	0.0000	-33.6098
104	2264	2265	8.4191	-25.1907
112	2265	2263	-16.8495	-42.0401
117	2263	2265	16.8495	-25.1907
118	2265	2265	0.0000	-25.1907
119	2265	2267	16.8049	-8.3858
119	2267	2263	-33.6543	-42.0401
120	2263	2264	8.4303	-33.6098
126	2264	2238	-222.8747	-256.4845
132	2238	2339	824.6632	568.1787
139	2339	2252	-703.6947	-135.5160
146	2252	2246	-51.5667	-187.0827
152	2246	2280	286.8529	99.7702
153	2280	2282	16.4755	116.2457
154	2282	2282	0.0000	116.2457
159	2282	2319	297.1503	413.3960
161	2319	2319	0.0000	413.3960
166	2319	2180	-1196.1104	-782.7144
168	2180	2155	-240.0361	-1022.7505
173	2155	2159	38.9668	-983.7836
175	2159	2161	19.4023	-964.3813
180	2161	2171	96.2087	-868.1726
182	2171	2170	-9.5611	-877.7337
187	2170	2196	244.3587	-633.3749
189	2196	2195	-9.2381	-642.6131
196	2195	2193	-18.5142	-661.1273
201	2193	2192	-9.2761	-670.4034
203	2192	2192	0.0000	-670.4034
208	2192	2194	18.5396	-651.8638
210	2194	2196	18.4889	-633.3749
215	2196	2194	-18.4889	-651.8638
217	2194	2192	-18.5396	-670.4034
221	2192	2191	-9.2888	-679.6922
223	2191	2195	37.0791	-642.6131
228	2195	2194	-9.2508	-651.8638
231	2194	2194	0.0000	-651.8638
238	2194	2193	-9.2634	-661.1273
269	2193	2190	-27.8665	-688.9938
275	2190	2190	0.0000	-688.9938
288	2190	2165	-236.7441	-925.7378
295	2165	2164	-9.6408	-935.3786
303	2164	2164	0.0000	-935.3786
309	2164	2163	-9.6542	-945.0328
314	2163	2163	0.0000	-945.0328

TABLE 2 : measured stress change by 1-25

TIME	TOT. STRESS CHANGE	TOT. STRESS CHANGE
DAYS	I-24	I-25
	PSI	PSI
28		
35	-25.8474	
42	-29.5548	16.7383
47	-33.2659	0
49	-40.6994	-8.3858
54	-40.6994	-8.3858
56	-36.9808	-16.7827
63	-55.6114	-33.6098
68	-51.8778	-33.6098
70	-78.0928	-33.6098
75	-55.6114	-33.6098
84	-63.0901	-33.6098
89	-66.8351	-25.1907
91	-66.8351	-33.6098
96	-66.8351	-33.6098
104	-63.0901	-25.1907
112	-63.0901	-42.0401
117	-48.1479	-25.1907
118	-104.4944	-25.1907
119	-100.7112	-8.3858
119	-96.932	-42.0401
120	-112.0722	-33.6098
126	-89.3849	-256.4845
132	-96.932	568.1787
139	-63.0901	-135.516
146	-66.8351	-167.0827
152	-11.0552	99.7702
153	-14.7476	116.2457
154	-14.7476	116.2457
159	40.2518	413.396
161	40.2518	413.396
166	-188.7044	-782.7144
168	-342.7312	-1022.7505
173	-255.085	-983.7836
175	-266.9199	-964.3813
180	-259.0259	-868.1726
182	-262.9709	-877.7337
187	-100.7112	-633.3749
189	-112.0722	-642.6131
196	-127.274	-661.1273
201	-138.716	-670.4034
203	-146.3635	-670.4034
208	-165.5505	-651.8638
210	-204.2195	-633.3749
215	-204.2195	-651.8338
217	-278.7915	-670.4034
221	-227.6116	-679.6922
223	-208.1082	-642.6131
228	-231.5243	-651.8638
231	-231.5243	-651.8638
238	-227.6116	-661.1273
269	-219.7982	-688.9938
275	-131.0841	-688.9938
288	-123.4678	-925.7378
295	-119.6654	-935.3786
303	-123.4678	-935.3786
309	-131.0841	-945.0328
314	-131.0841	-945.0328

TABLE 3 : measured stress change by I-24 and I-25.

DAYS	TOTAL DISPLACEMENT CH.#2, mm	TOTAL DISPLACEMENT CH.#3 , mm
28	0	0
35	0.35	0.12
42	0.52	0.19
47	0.65	0.19
49	0.7	0.21
54	0.75	0.21
56	0.8	0.22
63	0.84	0.24
68	0.85	0.26
70	0.87	0.27
75	0.87	0.27
84	0.88	0.28
89	0.88	0.28
91	0.88	0.28
96	0.87	0.28
104	0.87	0.28
112	0.14	0.48
117	0.12	0.49
118	0.1	0.49
119	0.1	0.5
119	0	0.51
120	-0.02	0.49
126	-0.52	0.47
132	-0.5	0.63
139	-0.83	0.84
146	0.19	1.37
152	0.25	1.52
153	0.25	1.62
154	0.25	1.62
159	0.24	1.8
161	0.26	1.8
166	0.23	2.08
168	0.23	2.22
173	.05	2.88
175	.04	2.89
180	0.03	2.89
182	0.01	2.87
187	-0.17	2.93
189	-0.24	2.93
196	-0.25	2.9
201	-0.26	3.03
203	-0.26	3.01
208	-0.27	3.11
210	-0.27	3.09
215	-0.26	3.11
217	-0.2	3.2
221	-0.21	3.12
223	-0.21	3.2
228	-0.21	3.14
231	-0.21	3.14
238	-0.21	3.15
269	-0.22	3.16
275	-0.21	3.16
288	-0.21	3.16
295	-0.2	3.18
303	-0.07	3.26

TABLE 4 : movement measured by extensometer G-13

DAYS	TOTAL DISPLACEMENT CH.#1, mm	TOTAL DISPLACEMENT CH. # 2, mm	TOTAL DISPLACEMENT CH. # 3, mm
28	0	0	0
35	-0.08	-0.09	-0.09
42	0.01	0.03	0.01
47	0.01	0.02	0
49	0.01	0.02	0
54	0.01	0.02	0
56	0.01	0.03	0
63	0	0.01	0.02
68	-0.01	0.02	0.02
70	-0.03	0	0.02
75	-0.03	-0.02	0.02
84	-0.03	0.01	0.01
89	-0.03	0.01	0.01
91	-0.03	0.01	0.01
96	-0.08	0.01	0.01
104	-0.16	0.01	0.01
112	-0.16	0.18	0.05
117	-0.16	0.18	0.05
118	-0.03	0.17	0.05
119	-0.03	0.19	0.04
119	1.23	0.21	0.05
120	1.27	0.92	0.27
126	0.76	0.95	0.29
132	3.46	1.05	0.53
139	3.34	4.34	1.81
146	3.35	6.17	2.87
152	3.34	6.29	2.84
153	3.27	6.56	2.84
154	3.26	11.16	2.27
159	3.28	11.55	2.23
161	3.36	15.24	1.96
166	3.33	19.47	1.14
168	3.35	21.1	1.13
173	3.34	21.13	1.13
175	3.33	21.49	1.13
180	3.33	21.61	1.12
182	3.31	26.07	1.05
187	3.31	26.62	1.06
189	3.33	27.07	1.03
196	3.33	27.61	1.04
201	3.33	27.62	1.04
203	3.32	27.8	1.05
208	3.31	27.93	1.05
210	3.3	27.87	1.04
215	3.29	28.31	1.04
217	3.28	28.63	1.04
221	3.32	28.78	1.04
223	3.28	29.03	1.06
228	3.28	29.13	1.04
231	3.25	29.7	1.04
238	3.29	29.04	1.04
269	3.28	30.72	1.02
275	3.28	30.94	1.02
288	3.29	32.16	1.01
295	3.08	33.47	0.94
303	3.08	31.98	-0.14
309	2.87	31.97	-0.09
314	2.93	32.01	0.24
321	2.93	32.01	0.23
334	2.86	32.01	0.21
341	2.86	32.01	0.21
349	2.86	32.01	0.2
356		32.01	0.2

TABLE 5 : movement measured by extensometer G-14

APPENDIX C

RESULTS OF SEISMOMETER TESTS AND BLAST VIBRATION MONITORING

AT

KIDD CREEK MINES LIMITED

Falconbridge Ltd. / 1990

Blast	Delay	Time	V4	V5	V6	PPV	V1	V2	V3	PPV	Comment
			mv	mv	mv	i.p.s	mv	mv	mv	i.p.s	**blast A**
A	1	41	50	87	48	3.9	27	22	16	1.3	Channels 4-6 in rock was
38-1	2	55	58	33	35	2.6	21	21	19	1.2	51 m from blast
	3	89	63	103	75	5	41	27	15	1.8	
	4	105	42	25	33	2.1	21	25	12	1.2	Channels 1-3 in backfill was
	5	136	38	78	41	3.4	31	18	15	1.4	53.2 m from blast
	6	169	20	38	16	1.6	12	20	12	0.9	
	7										
	8	230	67	87	70	4.6	32	32	16	1.7	
	9	283	37	48	42	2.6	15	23	12	1.1	
	10	345	51	76	66	4	43	32	19	2	
	11	411	24	29	21	1.5	14	17	8	0.8	
	12	510	59	113	64	5	34	21	15	1.5	
	13	588	18	44	9	1.7	10	11	11	0.6	
	14										
	15	771	57	79	29	3.8	17	26	13	1.2	
	16	821	45	75		3.2	17	29	12	1.3	**blast B:**
	17										Channel 4-6 in rock
	18	994	19	41	10	1.6	10	16	10	0.7	Channel 1-3 in fill and 40 m
	19	1051	35	82	48	3.6	15	18	10	0.9	from blast
	20	1176	23	43	10	1.7	9	20	9	0.8	**blast C:**
	21	1278	51	156	44	6	17	10	10	0.8	Channel 4-6 in rock 62.5 m
											from blast
B	1	28	67	29	53	3.2	24	26	32	1.7	Channel 1-3 in rock but
38-1	3	79	49	31	34	2.4	23	30	31	1.7	signal passing through back
											61.3 m from blast
C	1	37	123	82	48	2.7	101	54	76	2.4	
38-2											*PPV is in inches/sec.

TABLE 6 : peak particle velocity calculation, method #1

BLAST	DELAY	TIME	ch# 4	ch #5	ch#6	PPV , ROCK
38 - 1	NO.	mv	mv	mv	mv	ips
	1	41	30	92	28	3.5
	2	55	52	30	32	2.4
	3	89	70	115	64	5.2
	4	105	30	25	30	1.7
	5	136	40	102	62	4.4
	6	169	20	20	15	1.1
	7	--------	-------	-------	-------	------------
	8	230	82	118	45	5.3
	9	283	55	30	28	2.4
	10	345	52	60	70	3.7
	11	411	25	30	20	1.5
	12	510	108	150	68	6.9
	13	588	18	48	9	1.8
	14	--------	-------	-------	-------	------------
	15	771	52	102	25	4.1
	16	821	63	70	22	3.4
	17	--------	-------	-------	-------	------------
	18	994	25	82	41	3.3
	19	1051	44	82	40	3.5
	20	1176	22	40	12	1.65
	21	1278	40	138	40	5.2
						P.P.V =3.4
B- BLAST						
38 - 1	1	58	58	31	40	2.7
	3	50	50	31	28	2.3
						P.P.V= 2.5
C-BLAST						
38 - 2	1	PEAK TO PEAK	120	110	43	2.92
		37	121	100	40	2.82
						P.P.V= 2.9

TABLE 7 : peak particle velocity calculation, method # 2

ch #1	ch # 2	ch#3	PPV	PPV RATIO %	STRESS RATIO
mv	mv	mv	FILL, ips	FILL/ROCK	FILL/ROCK %
22	20	8	1.05	30.00	15.00
20	31	15	1.4	58.33	29.17
32	27	18	1.6	30.77	15.38
11	18	18	0.84	49.41	24.71
22	24	11	1.2	27.27	13.64
10	12	10	0.65	59.09	29.55
18	22	8	1	18.87	9.43
33	20	9	1.4	58.33	29.17
32	21	13	1.41	38.11	19.05
11	13	10	0.69	46.00	23.00
33	20	11	1.4	20.29	10.14
8	11	8	0.55	30.56	15.28
11	30	15	1.24	30.24	15.12
11	30	11	1.18	34.71	17.35
10	13	10	0.67	20.30	10.15
10	18	10	0.8	22.86	11.43
8	11	8	0.55	33.33	16.67
11	10	8	0.59	11.35	5.67
			P.P.V= 1 ips	34%	17%
20	21	18	1.2	44.44	22.22
21	21	30	1.48	64.35	32.17
			P.P.V= 1.34 ips	54%	27%
90	61	77	2.31	79.11	39.55
32.5	27	30	0.9	31.91	15.96
			P.P.V= 1.6 ips		

TABLE 8 : peak particle velocity calculation, method #2

SUPPLEMENT TO APPENDIX C

V_a = Measured vibration level in rock at a distance of 50 m (R_a) from the blast source = 2.8 i.p.s

V_b = Measured vibration level in fill for shock waves travelling the first 15 m (R_b) in rock and the other 35 m (R_c) in CRF.

V_c = Vibration level at 15 m from the blast in rock, as derived from :
$= V_a (R_a / R_b)^{1.8}$

V_d = Vibration level at the boundary, 15 m from the blast, in fill
$= 0.37\ V_c$ from the result of Blast A = 0.91 i.p.s

The seismic attenuation, constant in CRF, can be found from :
seismic attenuation $= 20 \log_{10} [(V_b / V_d)/ R_c] =$ **- 0.42 db/m**

APPENDIX D

DILATOMETER AND PRESSURE METER ANALYSIS

FOR

KIDD CREEK MINES LIMITED

Falconbridge Ltd. / 1990

TROW
DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº :- 2
TEST Nº : 3
DEPTH : 3'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

SURFACE ELEVATION :

TEST ELEVATION :

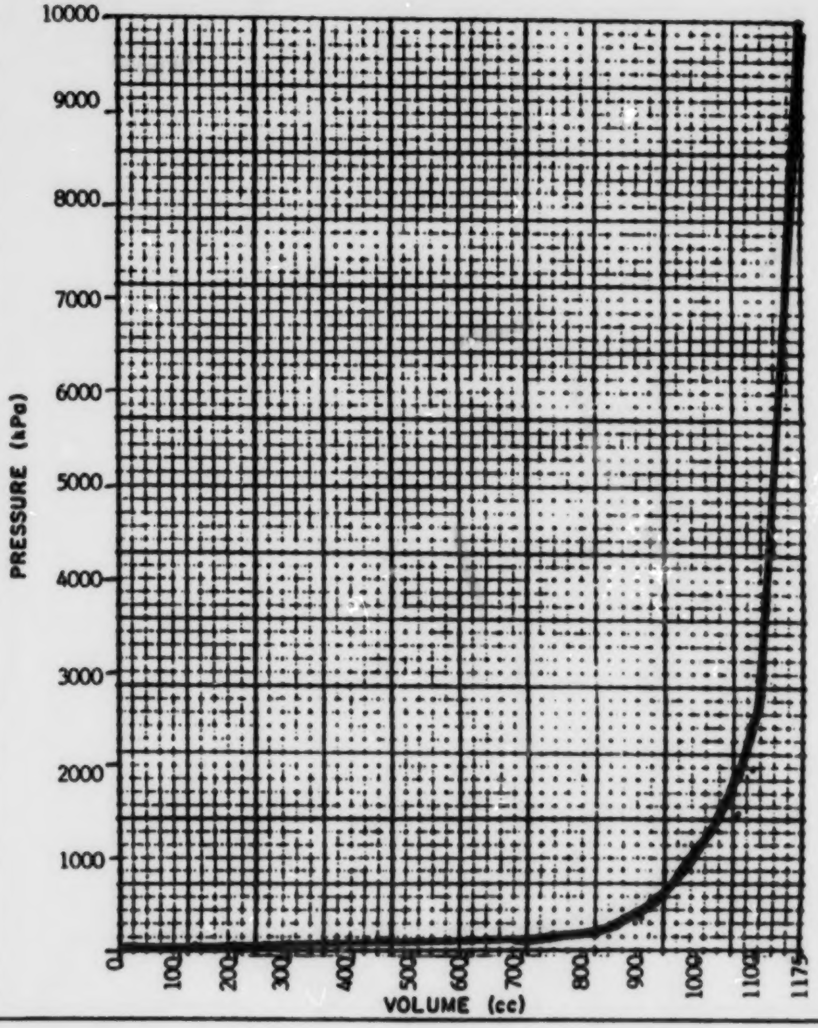
RESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
25				150	
30				200	
50				250	
60				300	
70				350	
75				400	
80				450	
100				500	
109				550	
112				600	
121				650	
126				700	

OLUMETRIC CALIBRATION :

BSERVATIONS : $u = .33$ $\Delta V = 1170 - 1098 = 72\ cc$ $\Delta P = 7000\ kPa$

$V_m = 1134\ cc$

$$E_\phi = 2.66\ (1270 + 1173.4) \left(\frac{1}{72/1000 - .84 \times 10^{-2}} \right) = 3.38\ GPa$$



TROW

DILATOMETER/PRESSUREMETER TESTS

SURFACE ELEVATION :

TEST ELEVATION :

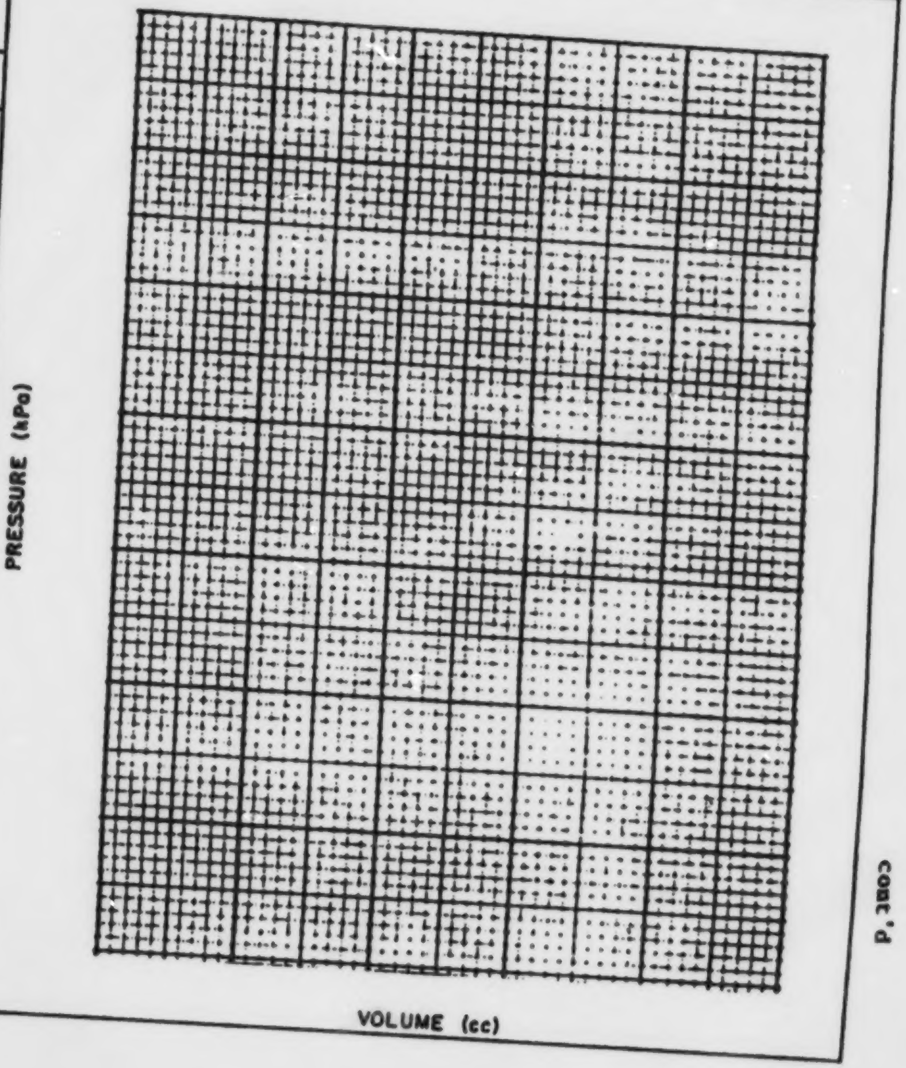
BOREHOLE Nº : 2 (cont'd)
TEST Nº : 3
DEPTH : 3'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

RESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
150				750	
175				800	
230				850	
330				900	
500				1061.9	
000				1096.8	
000				1098	
000				1109.1	
000				1119.2	
000				1129.8	
000				1138.7	
000				1151.6	

OLUMETRIC CALIBRATION :

BSERVATIONS :



cont'd

cont'd

TROW
DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)
TEST Nº : 3
DEPTH : 3'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

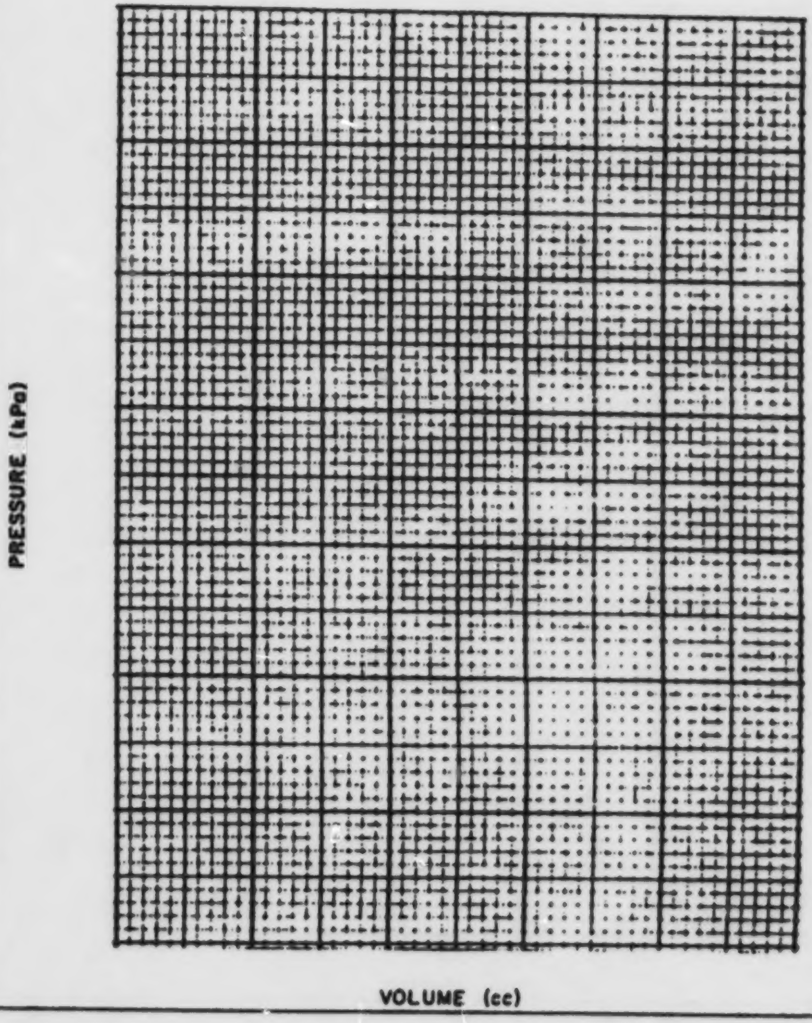
SURFACE ELEVATION :

TEST ELEVATION :

RESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
9000				1160.6	
10000				1170.0	

OLUMETRIC CALIBRATION :

OBSERVATIONS :



TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº :- 2
TEST Nº : 2
DEPTH : 6'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

SURFACE ELEVATION :

TEST ELEVATION :

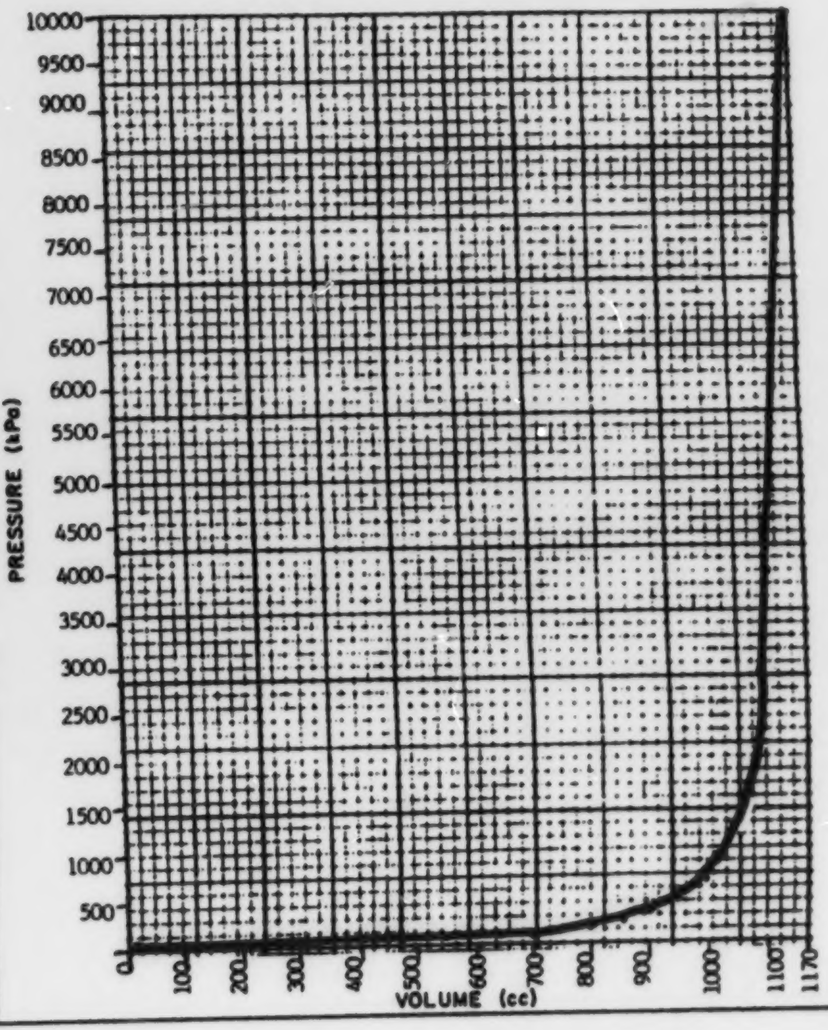
RESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
0				50	
25				100	
30				150	
50				200	
60				250	
70				300	
78				350	
87				400	
98				450	
106				500	
112				550	
120				600	

OLUMETRIC CALIBRATION :

IBSERVATIONS : $u = .33$ $\Delta V = 1164.6 - 1075 = 89.6$ cc $\Delta P = 8000$ kPa

$V_m = 1119.8$ cc

$ɇ = 2.66\ (1270 + 1119.8) \left(\frac{1}{89.6/8000 - .84 \times 10^{-2}} \right) = 2.27$ GPa.



TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)
TEST Nº : 2
DEPTH : 6'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

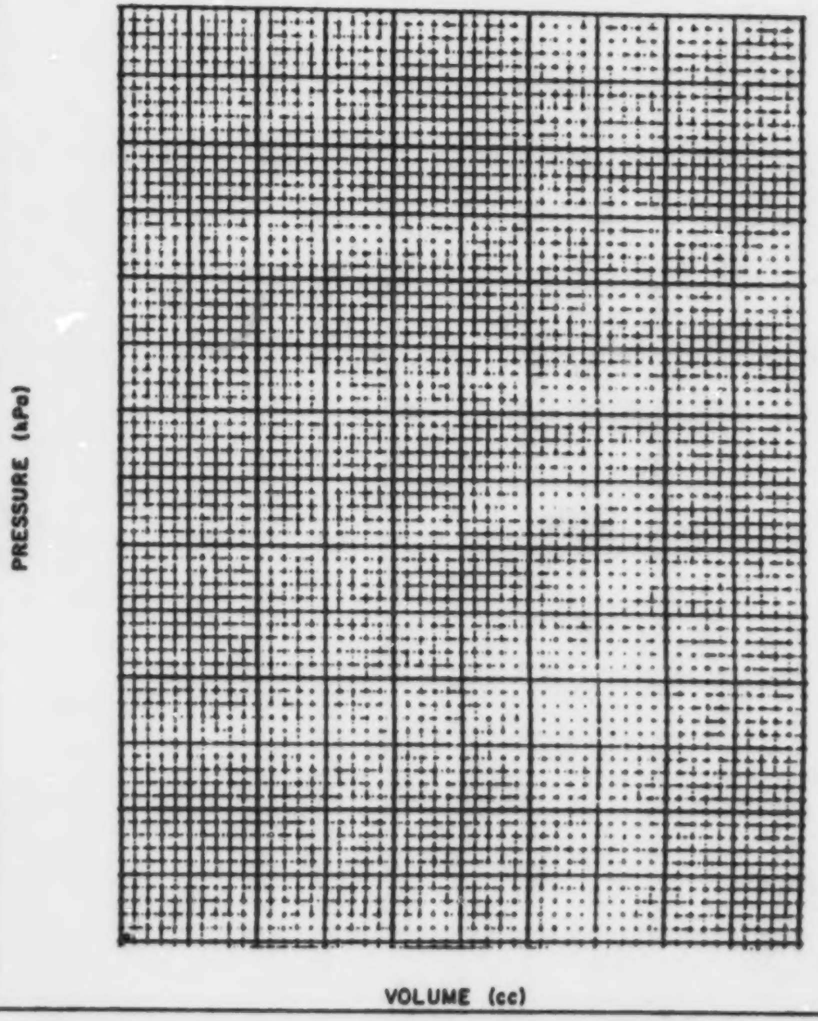
SURFACE ELEVATION :

TEST ELEVATION :

PRESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
125				650	
130				700	
150				750	
175				800	
230				850	
325				900	
430				950	
675				1000	
000				1035.7	
2000				1075	
3000				1092.6	
000				1104.8	

VOLUMETRIC CALIBRATION :

OBSERVATIONS :



cont'd

TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)		**PROJECT Nº** :	S03365R
TEST Nº : 2		**SITE** :	Kidd Creek Backfill
DEPTH : 6'			
HOLE SIZE : NW			

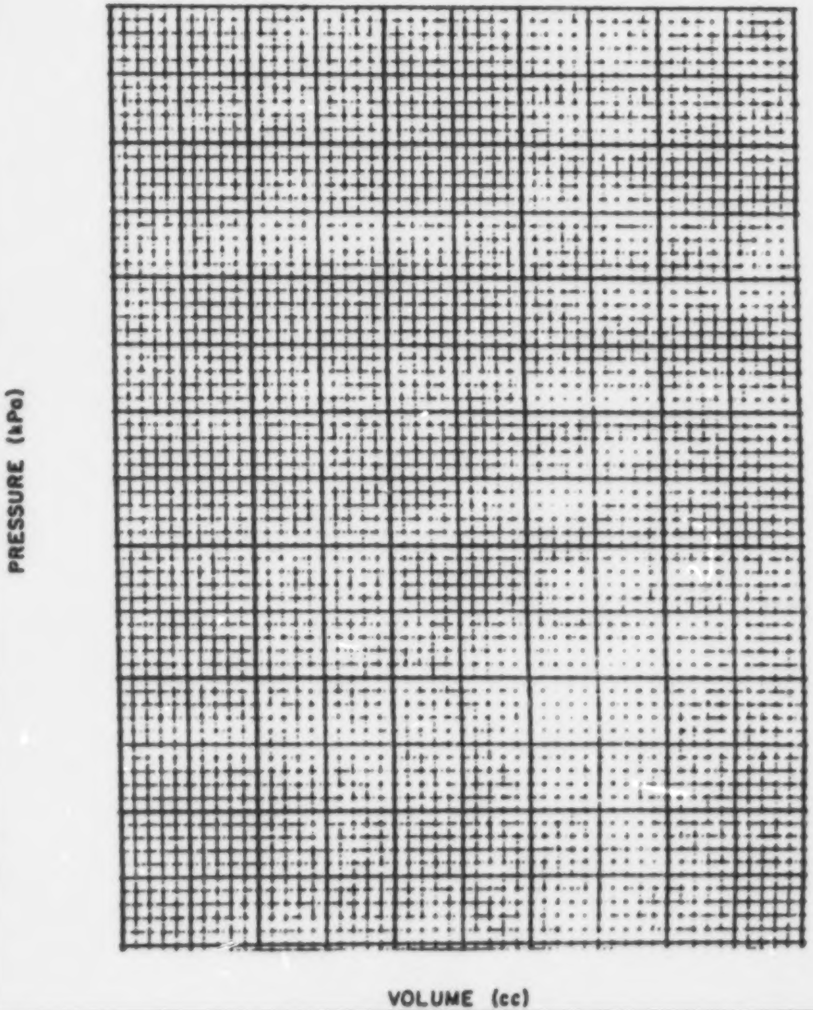
SURFACE ELEVATION :

TEST ELEVATION :

'RESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
5000				1116.9	
6000				1126.1	
7000				1135.6	
8000				1145.1	
9000				1154.4	
0000				1164.6	

OLUMETRIC CALIBRATION :

)BSERVATIONS :



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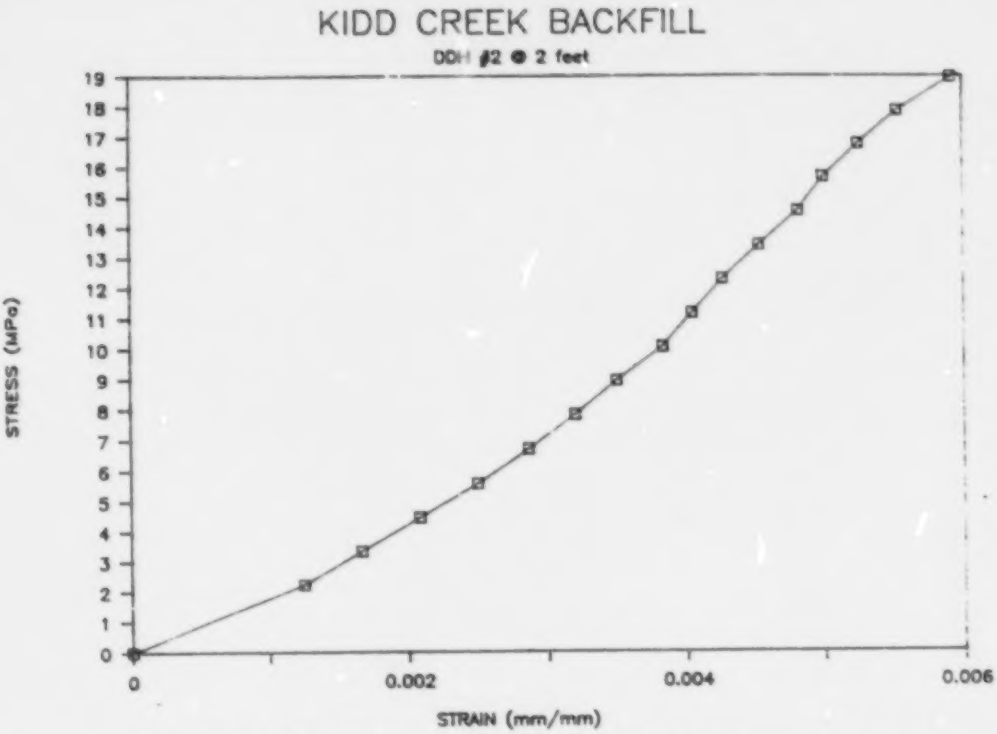


Figure: 7.3.2 : stress/ strain relationship of a CRF core sample.

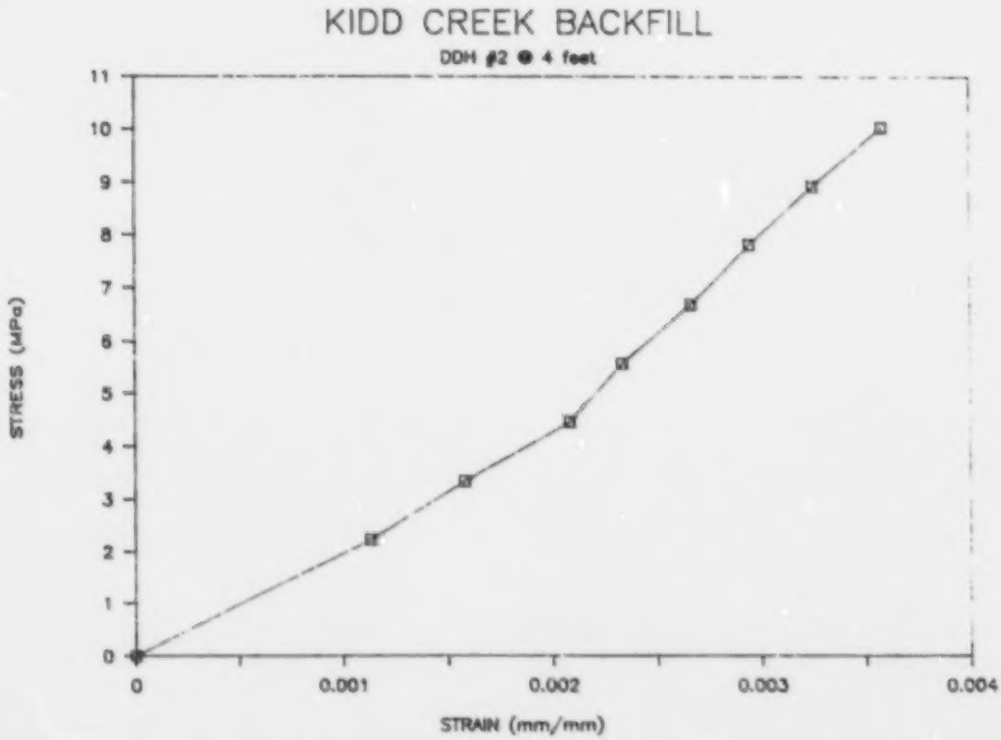


Figure: 7.3.3 : stress/ strain relationship of a CRF core sample.

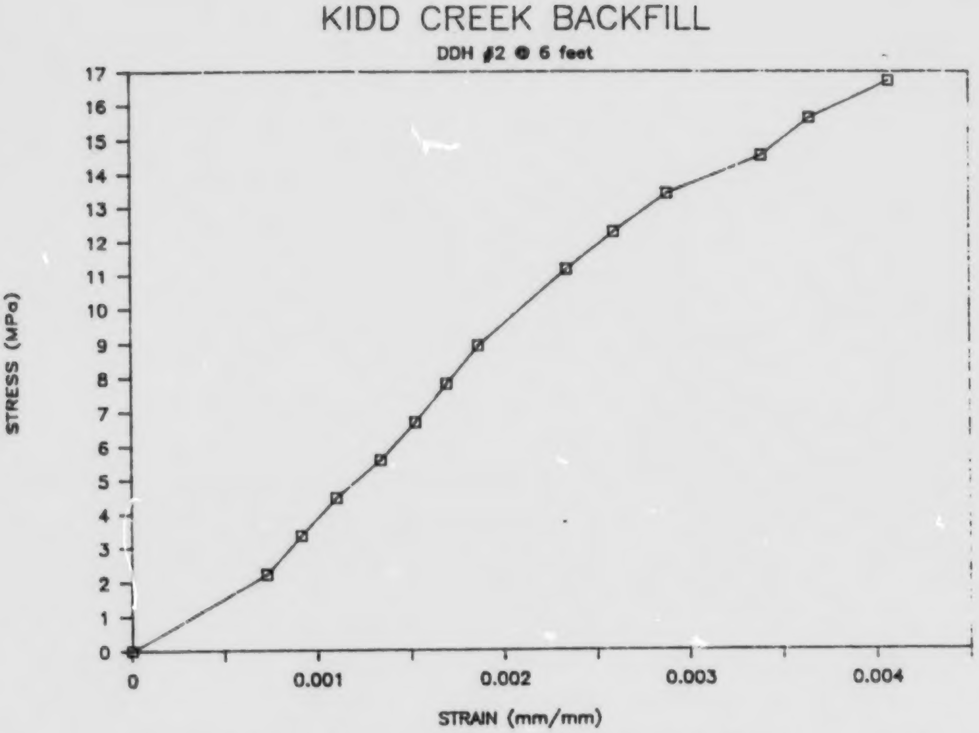


Figure: 7.3.4 : stress/ strain relationship of a CRF core sample.

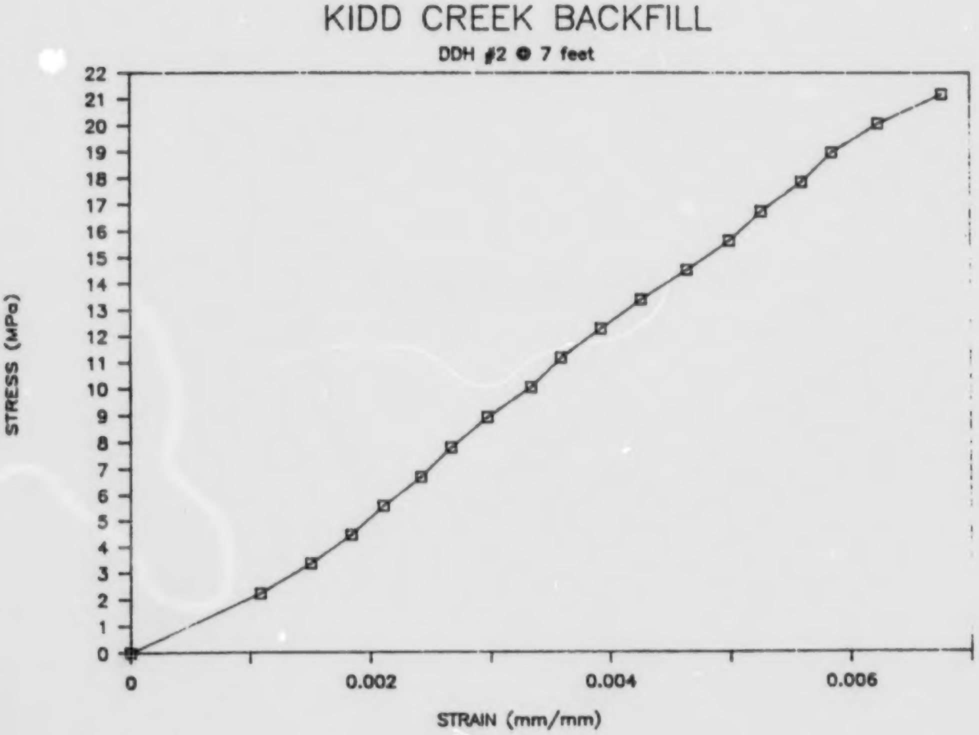


Figure: 7.3.5 : stress/ strain relationship of a CRF core sample.

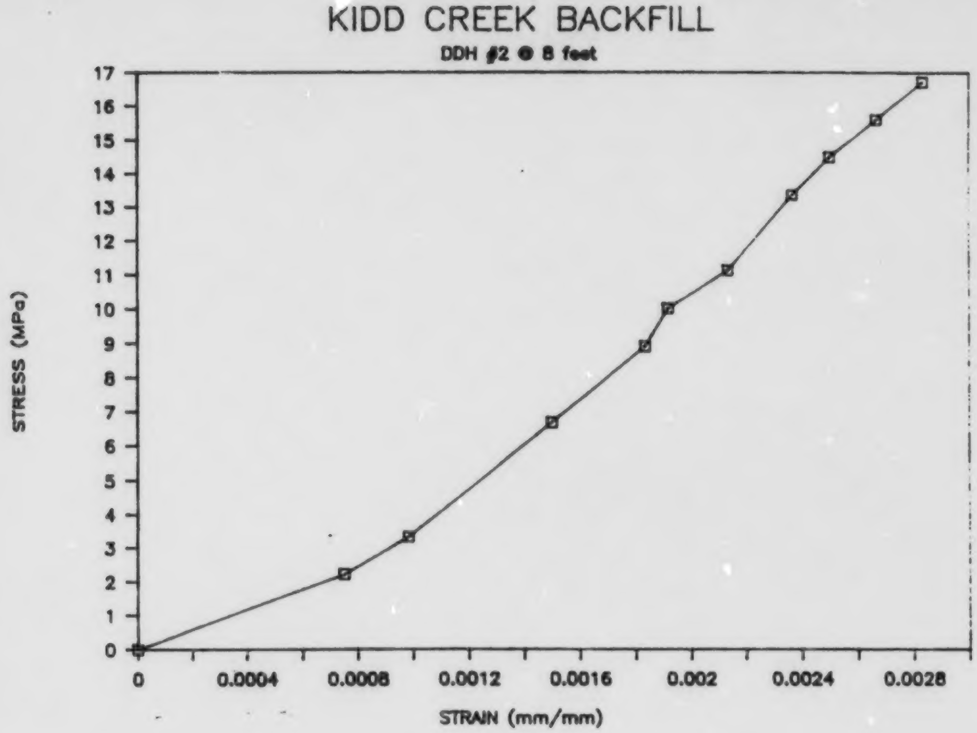


Figure: 7.3.6 : stress/ strain relationship of a CRF core sample.

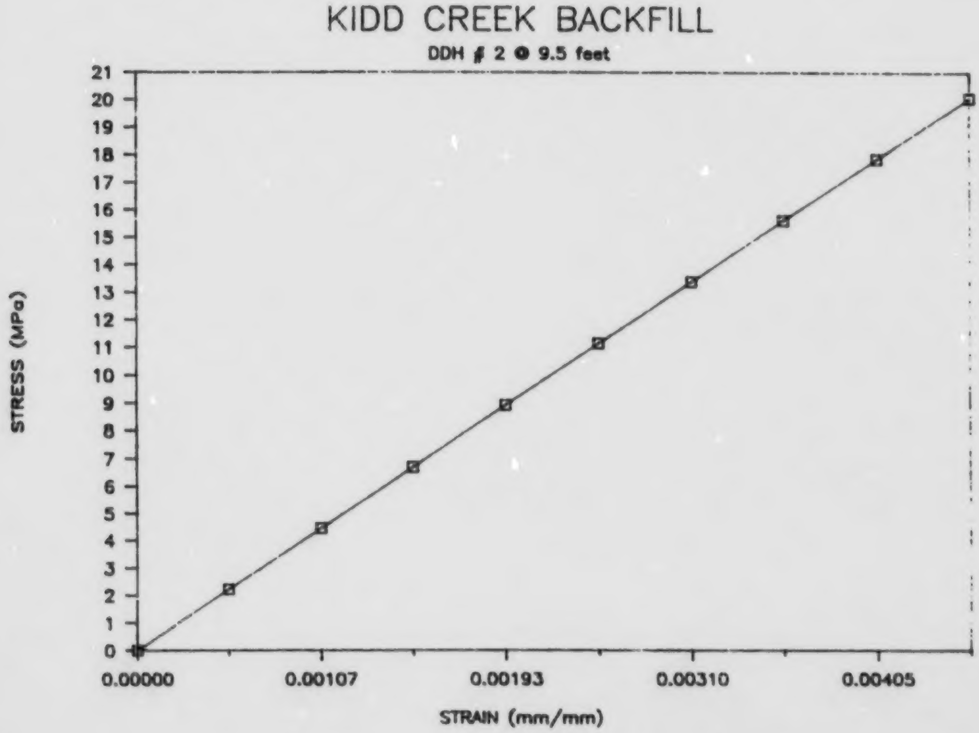


Figure: 7.3.7 : stress/ strain relationship of a CRF core sample.

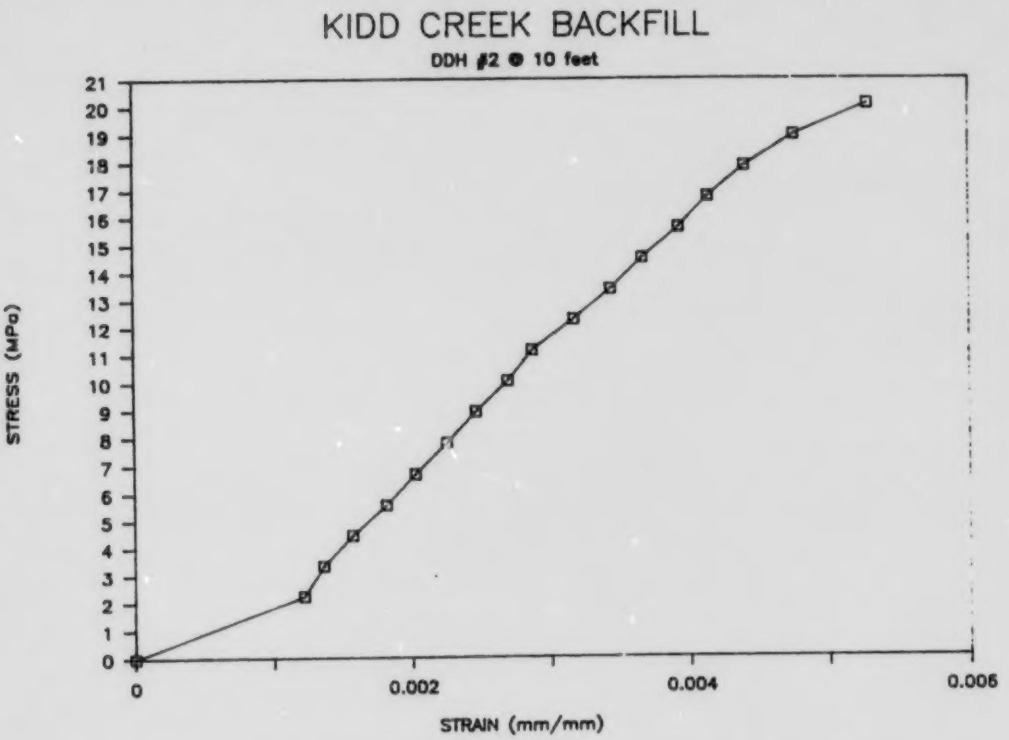


Figure: 7.3.8 : stress/ strain relationship of a CRF core sample.

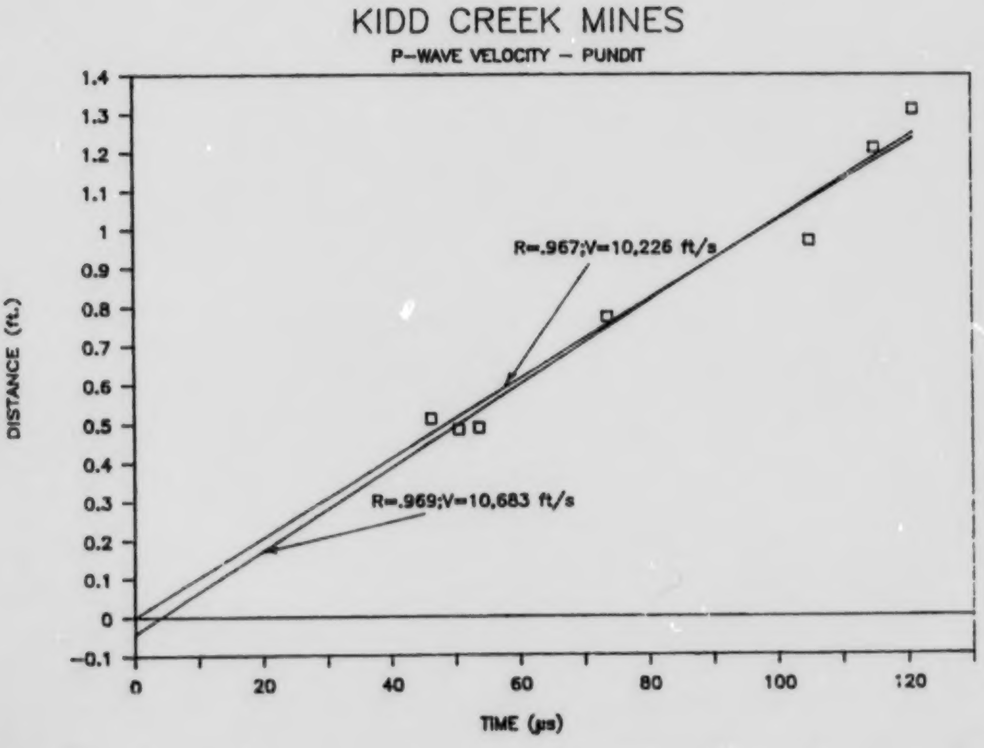


Figure: 7.3.9 : stress/ strain relationship of a CRF core sample.

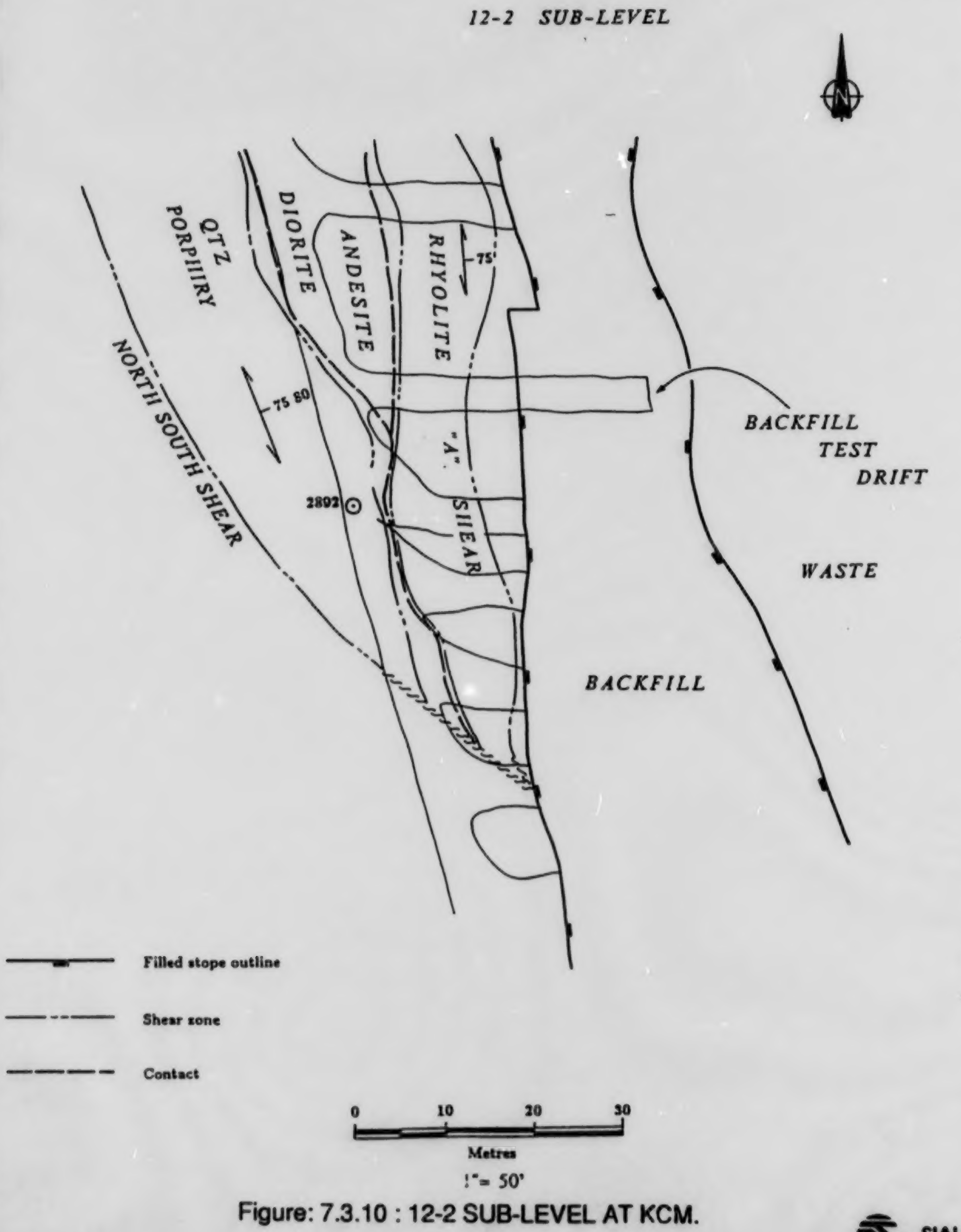
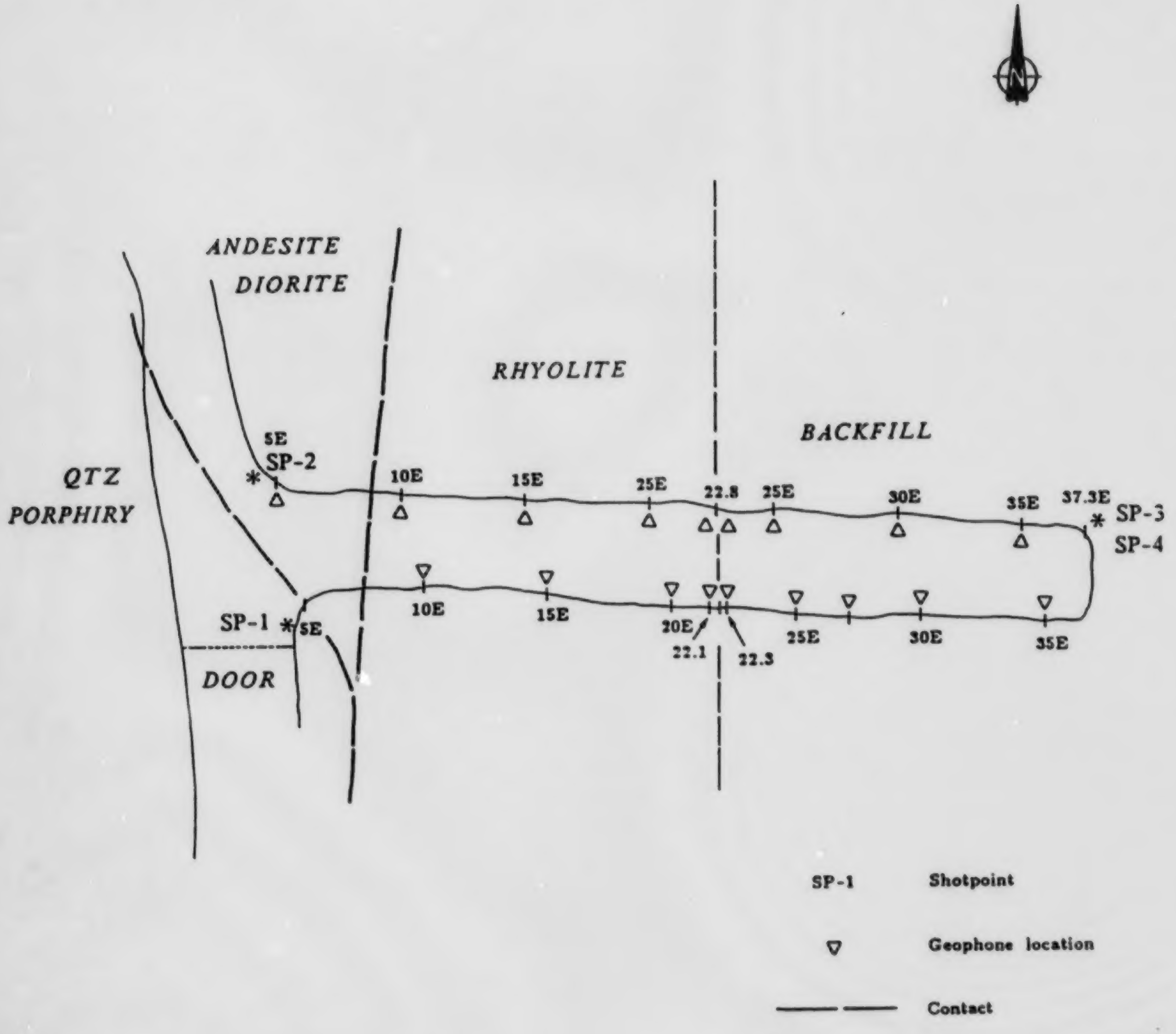

12-2 SUB-LEVEL
N
QTZ PORPHYRY
DIORITE
ANDESITE
RHYOLITE
75°
NORTH SOUTH SHEAR
75 80
"A" SHEAR
2892
BACKFILL TEST DRIFT
WASTE
BACKFILL
Filled stope outline
Shear zone
Contact
0
10
20
30
Metres
1"= 50'

Figure: 7.3.10 : 12-2 SUB-LEVEL AT KCM.

12-2 CRF Test Drift



N
ANDESITE
DIORITE
RHYOLITE
BACKFILL
QTZ
PORPHIRY
DOOR
5E
SP-2
10E
15E
25E
22.8
25E
30E
35E
37.3E
SP-3
SP-4
SP-1
5E
10E
15E
20E
22.1
22.3
25E
30E
35E
SP-1
Shotpoint
Geophone location
Contact

Figure: 7.3.11 : 12-2 TEST DRIFT AT KCM.

12-2 CRF Test Drift
SP-1 South Wall (near 5E)

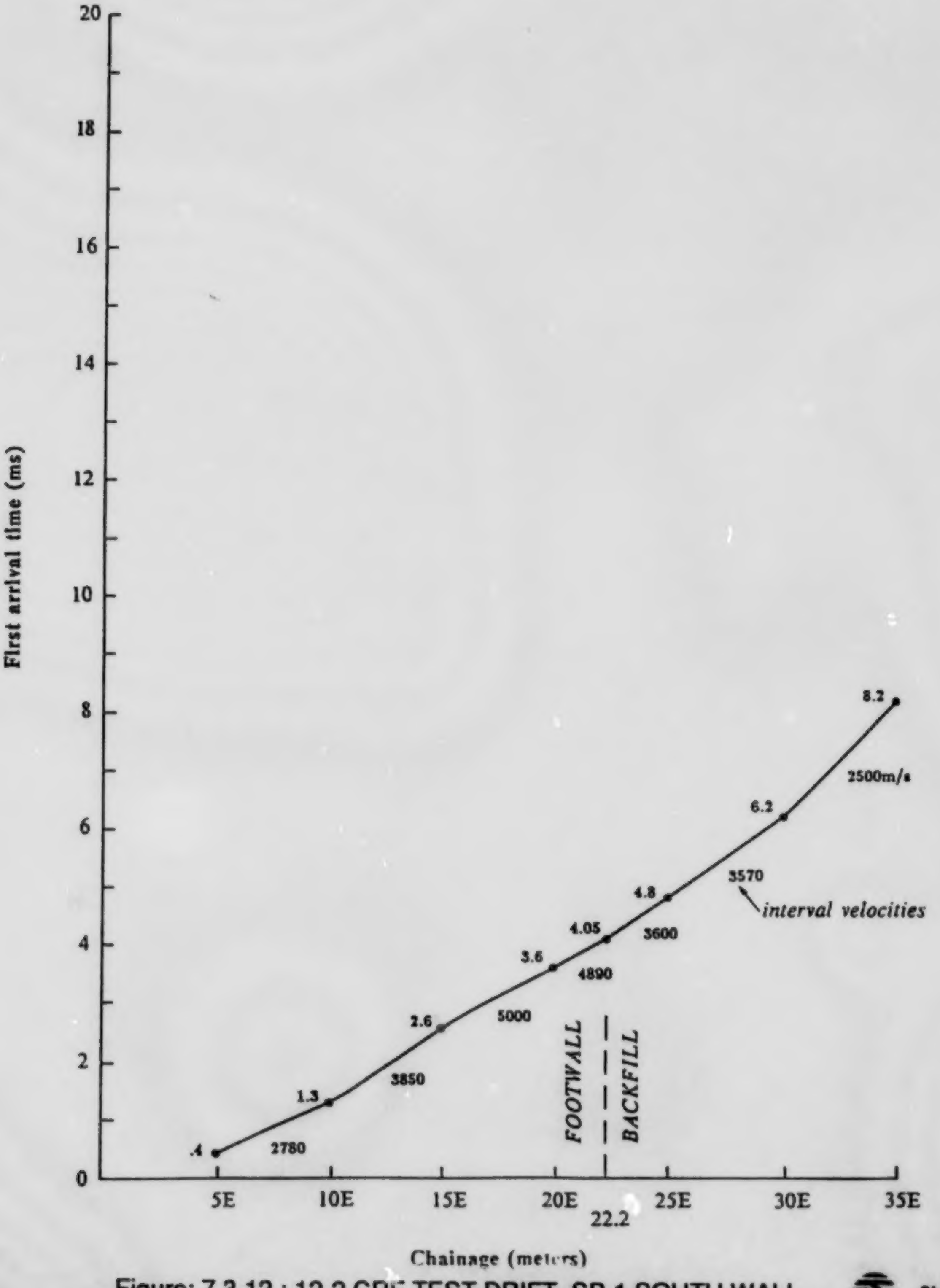


Figure: 7.3.12 : 12-2 CRF TEST DRIFT, SP-1 SOUTH WALL.

SIAL

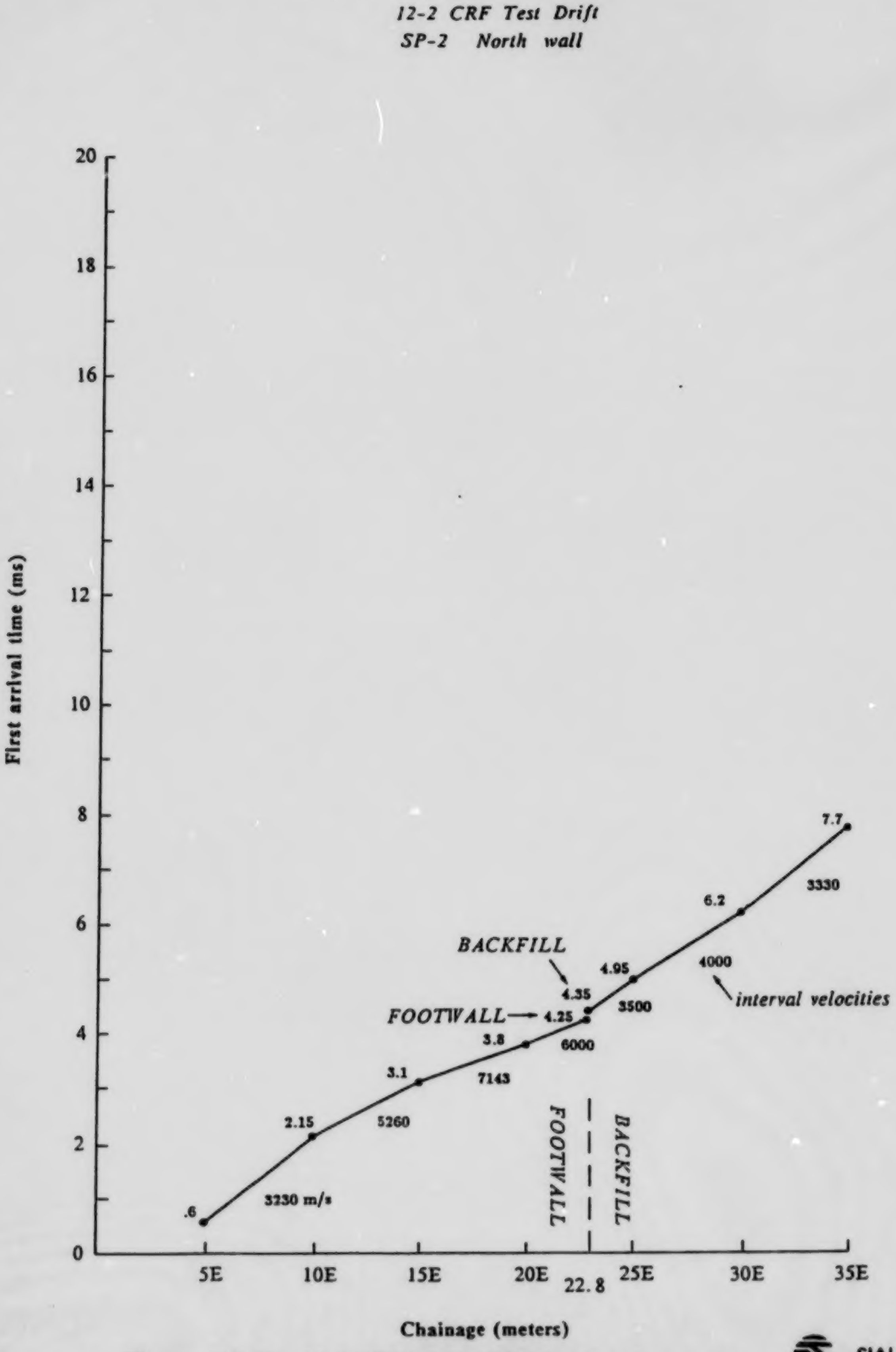
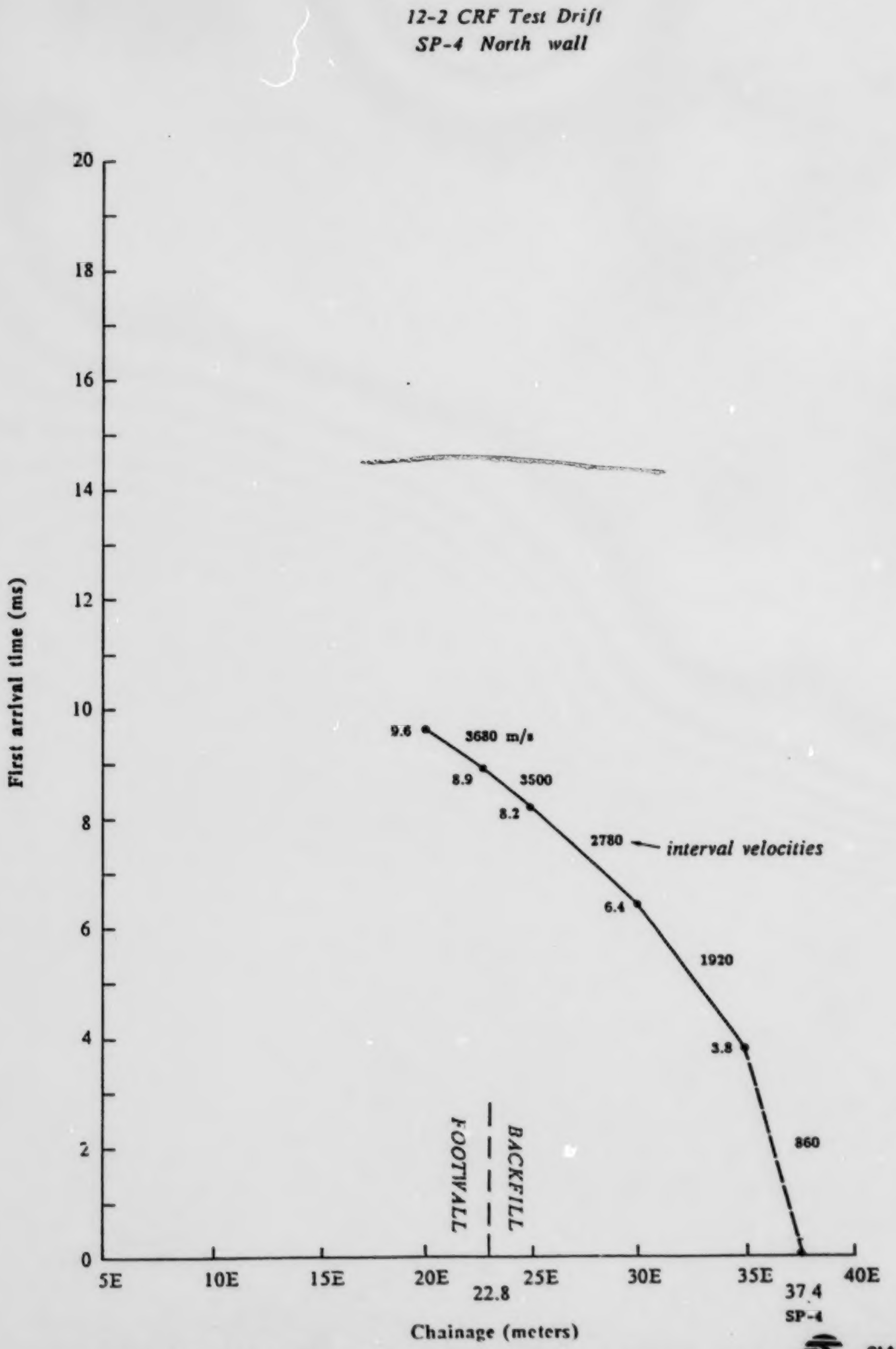


Figure: 7.3.13 : 12-2 CRF TEST DRIFT, SP-2 NORTH WALL.

SIAL



SIAL

Figure: 7.3.14 : 12-2 CRF TEST DRIFT, SP-4 NORTH WALL.

APPENDIX E

CENTRIFUGE MODELLING OF REINFORCED FILL

BY

KINGSTON GEORESEARCH

CENTRIFUGE MODEL STUDIES
ON
REINFORCED BACKFILLS

Phase 3 of subcontract on

Backfill Alternatives

Prepared for

FALCONBRIDGE LTD.

by

Robert J. Mitchell
Kingston GeoResearch Ltd.
August 1987

CENTRIFUGE MODEL STUDIES ON REINFORCED BACKFILLS

Kingston, Ontario 4 August 1987

ABSTRACT

Classified tailings from Falconbridge Ltd. Sudbury Operations were used to pour 0.3 m high models of 50:1 and 33:1 T:C backfills containing either geogrid reinforcements or mesh elements. Twelve models were tested in the centrifuge as a preliminary physical model study of the potential for using reinforcing elements to improve the performance characteristics of weakly cemented mine backfills. The results indicate that mesh elements are not likely to be economical in bulk pours because of costs, even though these elements have beneficial effects. Geogrid reinforcements, however, are shown to produce up to a seven fold increase in the critical height of a weakly cemented backfill exposure and could be useful in operations where stope access is available for placement of the reinforcing materials.

INTRODUCTION

The use of geo-reinforcements in Civil Engineering practice has been actively researched and promoted for the post decade but one of the earliest research studies on the potential for reinforcements in mine backfills was published by Mitchell and Stone, in 1987. These authors concluded that there was potential for further research on reinforced mine backfills. As part of a centrifuge model study program on backfill alternatives, twelve models were tested with commercial geogrid and mesh element reinforcements. The models were constructed using classified tailings from Falconbridge Ltd. Sudbury Operations and having an effective grain size of 0.01 mm and a uniformity coefficient of 13. Six models contained SS-1 geogrid reinforcements and six contained mesh elements commonly called ‘snippets’. Both types of reinforcements were provided, for research purposes, by the Tensor Corporation.

MODEL PREPARATION AND TESTING

Centrifuge models were prepared by pouring the design mixes into treated plywood molds. After curing in a moist room for 28 days, the models were transferred to centrifuge strongboxes. As concluded by Mitchell and Stone (1987), reinforcing of cemented mine backfills would only be useful in situations where the exposed fill strike length was greater than the exposed height, such situations being referred to as the two-dimensional case. To produce a two-dimensional or plane stress case, the centrifuge models were designed to fit into the strongboxes with a small end clearance of about 0.5 mm such that no significant boundary friction could develop during the model test. The models were nominally 0.2 m in strike length, 0.3 m in height and 0.1 m in width.

Mesh reinforcing elements (snippets) are nominally 50 mm by 100 mm in overall dimensions and are composed of a 5 x 10 grid of 0.5 mm thick plastic strands. When mixed with the fill pour, these inclusions alter the mass properties of the fill and do not need to be modelled providing the model dimensions are large with respect to the inclusion dimensions. In this case it was considered necessary to cut the snippets in order that they might achieve a distribution, within the fill, that would not be unduly affected by the model size. The snippets were cut to form 3 x 6 grids approximately 20 mm x 50 mm in size as shown on Figure 1(a). These were mixed and poured with the fill to form models at various concentrations of inclusions.

Geogrid SS-1 samples are conveniently produced to the exact dimensions of the model cross-section. These were placed at 3, 5 and 9 reinforcements per model according to the patterns shown on Figure 1(b). It is realized that a constant spacing may not be efficient but the main purpose of this preliminary study was to observe the mechanisms of failure and measure the general improvement due to reinforcements. For the same reason, no attempt was made to design the rupture strength, displacements nor pullout resistance of the reinforcements to any particular prototype scale. Some preliminary tests, using mesh elements to form continuous layers of reinforcing showed that these model reinforcements on 50 mm model spacings, did rupture at prototype heights of about 30 m. The SS-1 geogrid, being a prototype reinforcement, is relatively strong and was used to ensure that rupture would not occur. The geogrid reinforcement models were poured in layers with the geogrid sample being placed after the surface water had drained from each layer.

Model testing was carried out by conditioning the samples for a few minutes at a centrifuge speed below 100 RPM and then increasing the speed at a rate of about 4 RPM per minute until failure occurred. Smaller increases in centrifuge speeds were applied as failure was approached and the model was allowed 4 to 5 minutes to reach equilibrium at each speed (at each stress level).

MODEL TEST RESULTS

Figure 2 shows the relationship between critical height and unconfined compressive strength (UCS) as established from centrifuge model tests on plain cemented backfills with free end conditions. This relationship is linear with a stability number of 3.7, comparable to the theoretical upper bound value of 4.0 and the commonly used value of 3.8 for vertical slopes in cohesive soils. The value of 3.7 indicates a greater effect of vertical crack propagation in cemented backfill than in cohesive soils. The relationship between critical height and cement content is also plotted on Figure 2 and is seen to be of the form exected for cemented fills.

Table 1 lists the centrifuge test results. From these data it is clear that both types of reinforcement produce significant improvements in fill stability above the 7 m and 14 m critical heights for the plain 50:1 and 33:1 T:C backfills.

Figure 3 shows the failures developed in models A, B, C and D. With the greater spacings between geogrid layers, mass failure developed after initial basal spalling. With a reduced spacing, failures were limited to facial spalling, the depth of the spalling being restricted by the geogrid spacing. No pullout of the geogrid was evident.

Partially embedded mesh elements can be seen after the mass failure of model D. Greater concentrations of mesh elements resulted in some reduction of the volume of failed material as shown by models E and F on Figure 4.

Figure 4 shows the failures in models G and H. A SS-1 geogrid sample is shown above sample G. At the greater spacings, mass failure is facilitated by the flexibility of the geogrid and developed as follows: basal spalling in the lower fill layer occurred at 60% to 70% of the critical height, allowing some downward bending to develop in the exposed geogrid; at an increased centrifuge speed (increased stress level), a spalling type of failure occurred in the material above the exposed geogrid and this failure extended further into the fill due, apparently, to the rotation of the geogrid. Subsequent failures, at higher stress levels, were usually shear failures, resulting in a significant mass of material sliding out from the fill face. Mass failure developed above the second reinforcing level in model H shown on Figure 4, although permanent bending of the geogrid is not apparent in this photograph. Permanent bending of a geogrid reinforcement can be seen in model A on Figure 3. With relatively close geogrid spacings, end rotation in the geogrid was virtually eliminated and failure was limited to near surface facial spalling as seen in model I on Figure 5. In this model, initial base spalling occurred at an equivalent prototype height of about 40 m and had extended to the level shown in the photograph at an equivalent prototype height of about 60 m. In fact, models C and I never did fail, in the usual sense, and the amount of dilution produced by facial spalling was only about 4% of the fill mass at the defined critical heights.

Models J, K and L all exhibited mass failures as shown on Figure 5. The main problems with bulk pouring of fill mixed with mesh elements is that the mesh elements tend to agglomerate and may become aligned in directions where they contribute very little to fill stability.

It will be noted, from Table 1, that the control testing associated with 50:1 T:C models gave very consistent strength data but most of the measured strengths for the 33:1 T:C models are considerably less than the 150 kPa value given on Figure 2. The reason for this anomaly has not been ascertained but it is thought to be due to problems that developed with the moist room during the early curing stage of these tests. In order to compare the results for the two T:C mixtures, the equivalent prototype heights for the 33:1 T:C models have been corrected, on Table 2, to the average UCS of 150 kPa. Improvement factors, listed on Table 2, indicate the factor by which the critical height has been increased above the unreinforced values of 7 m and 14 m for the 50:1 and 30:1 T:C, respectively. The results are quite consistent.

It can be concluded that the mesh elements (snippets) do not produce benefits that exceed the costs of these elements -- 3 kg/m^3 of snippets would cost about $3/tonne of fill; for this amount, the cement content of the fill could be increased by 3 to 4%, producing an increase in critical height of fill that would be greater than that attributed to this concentration of snippets. The improvement due to horizontally placed geogrid reinforcements are quite substantial but the economics are not clear at this time. It should be noted that, while prototype stresses are reproduced in centrifuge models, prototype forces are not. Thus, there is no guarantee, from these model tests, that the geogrid reinforcements are adequate in terms of either pullout or rupture. From the model test results, the average prototype critical fill thickness per geogrid layer appears to vary from about 7 m for the 50:1 T:C fill to about 10 m for the 33:1 T:C fill. Using the evaluation approach outlined by Mitchell and Stone (1987), it would appear that the geogrid SS-1 is adequate at these spacings. For uncemented fill, the geogrid spacing would likely have to be reduced to between 3 and 5 meters but the cost advantages would be much greater if cement was eliminated entirely. Geogrid placement costs are quite site specific. In most high level blast hole stoping operations, the cost of making stopes safe for entry by men and equipment would be prohibitive. Fortunately, these are the types of operations where fill arching predominates the fill behaviour and reinforcements would not be very effective. In certain room and pillar operations, the pillars and backs are quite stable during primary mining and limited bolting could provide safety for working in the stopes. In such cases, extensive stope lengths can be mined out and reinforced primary backfills are likely to be economical. Two aspects of the geogrid systen have not been addressed in this study -- creep deformations and fire safety considerations. These may have significant effects on the usefulness of this particular system as a reinforcement for underground backfilling in mines.

CONCLUSIONS AND RECOMMENDATIONS

A major conclusion from the physical model studies reported herein is that both bulk poured mesh elements and horizontally placed geogrid reinforcements can produce significant improvements in the stability of weakly cemented mine backfills. Bulk pouring of mesh reinforcements is not likely to be economical, however, and further research on bulk poured inclusions should be directed to developing an anchored fibre type of inclusion, as suggested by Mitchell and Stone (1987).

The mmajor improvement created by placing geogrid reinforcements could be economically advantageous in certain room and pillar operations but is not useful in most blast hole stoping operations. The fact that fill movements were restricted to facial sloughing by relatively closely spaced geogrids sponsors the recommendation that geogrid reinforced uncemented fills be studied using physical models. Elimination of cement would release greater resources for geogrid installation and minor dilution with uncemented fill would generally be acceptable. Further model tests to determine optimum spacings and properties of reinforcements should also be carried out to provide data for a more realistic costing of this backfill alternative.

REFERENCES

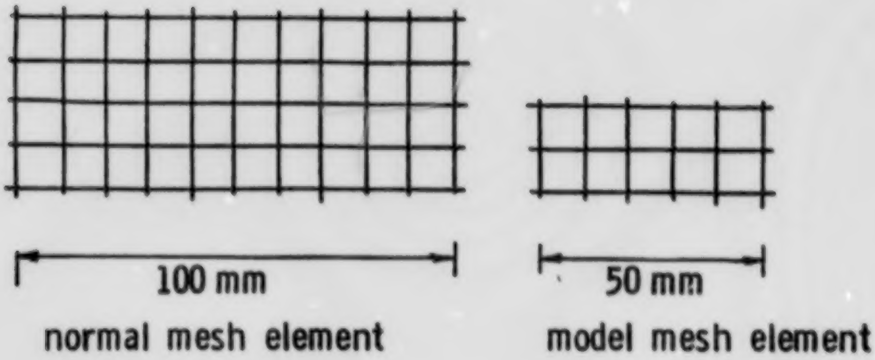
Mitchell, R.J. and Stone, D.M. (1987). Stability of Reinforced Cemented Backfills, Canadian Geotechnical Journal 214:2:189-197.

TEST NO.	SAMPLE DESCRIPTION	UCS OF CONTROL CYLINDERS kPa	CENTRIFUGE RPM AT FAILURE	EQUIVALENT PROTOTYPE HEIGHT m
A	50:1 T:C 3 layers geogrid	78	160	23
B	50:1 T:C 5 layers geogrid	78	185	31
C	50:1 T:C 9 layers geogrid	74	240	52
D	50:1 T:C 1 kg/m^3 snippets	74	108	11
E	50:1 T:C 2 kg/m^3 snippets	78	128	15
F	50:1 T:C 3 kg/m^3 snippets	78	160	23
G	33:1 T:C 3 layers geogrid	101	162	24
H	33:1 T:C 5 layers geogrid	100	218	43
I	33:1 T:C 9 layers geogrid	118	260	61
J	33:1 T:C 1 kg/m^3 snippets	166	157	22
K	33:1 T:C 2 kg/m^3 snippets	100	138	17
L	33:1 T:C 3 kg/m^3 snippets	100	175	28

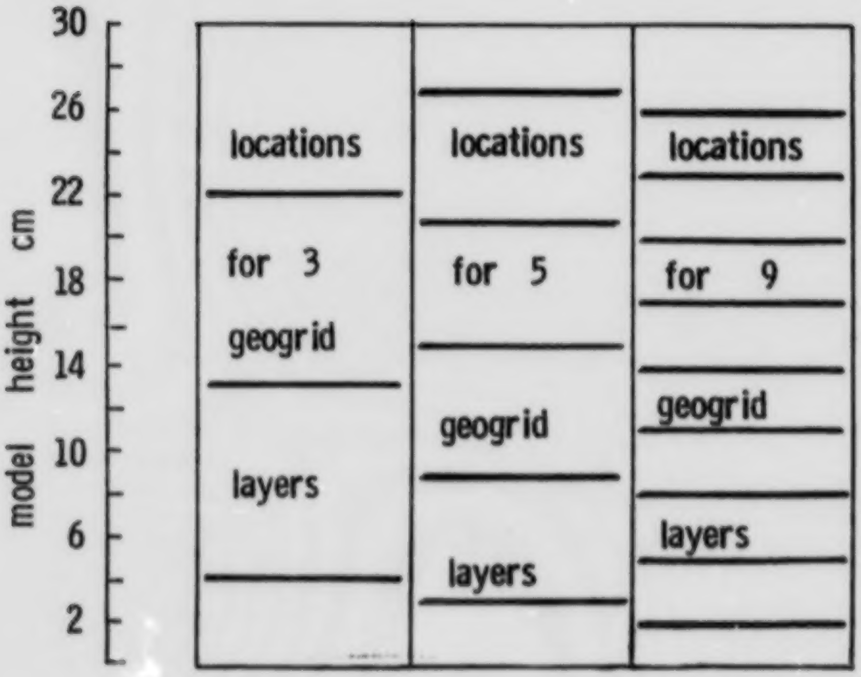
TABLE 1 CENTRIFUGE MODEL DATA

TEST NO.	T:C RATIO	REINFORCEMENT	CORRECTED PROTOTYPE HEIGHT, m	IMPROVEMENT FACTOR
A	50:1	3 layers of geogrid	23	3.3
G	33:1		36	2.7
B	50:1	5 layers of geogrid	31	4.4
H	33:1		65	5.0
C	50:1	9 layers of geogrid	52	7.4
J	33:1		78	6.0
D	50:1	1 kg/m^3 of snippets	11	1.5
J	33:1		20	1.5
E	50:1	2 kg/m^3 of snippets	15	2.1
K	33:1		26	2.0
F	50:1	3 kg/m^3 of snippets	23	3.3
L	33:1		42	3.2

TABLE 2 IMPROVEMENT IN CRITICAL HEIGHT DUE TO REINFORCEMENTS


100 mm
normal mesh element
50 mm
model mesh element

(a) mesh element reinforcements


30
26
22
18
14
10
6
2
model height cm
locations
for 3
geogrid
layers
locations
for 5
geogrid
layers
locations
for 9
geogrid
layers

(b) geogrid reinforcements

FIGURE 1 TYPES AND LOCATIONS OF REINFORCEMENTS

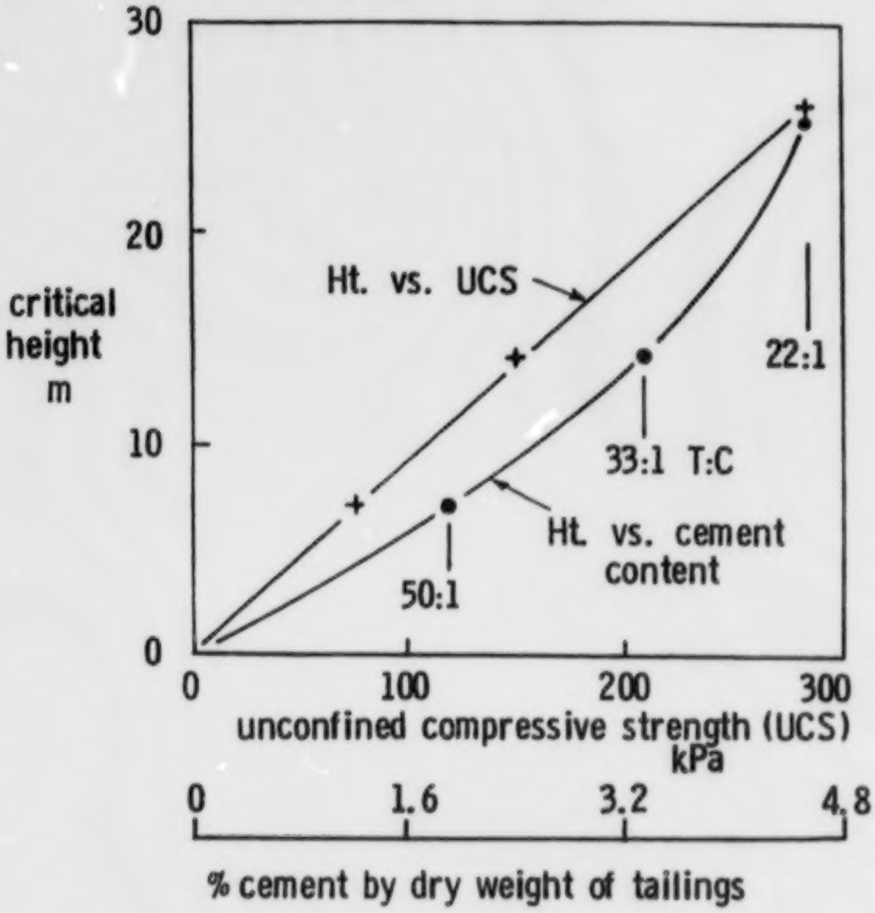
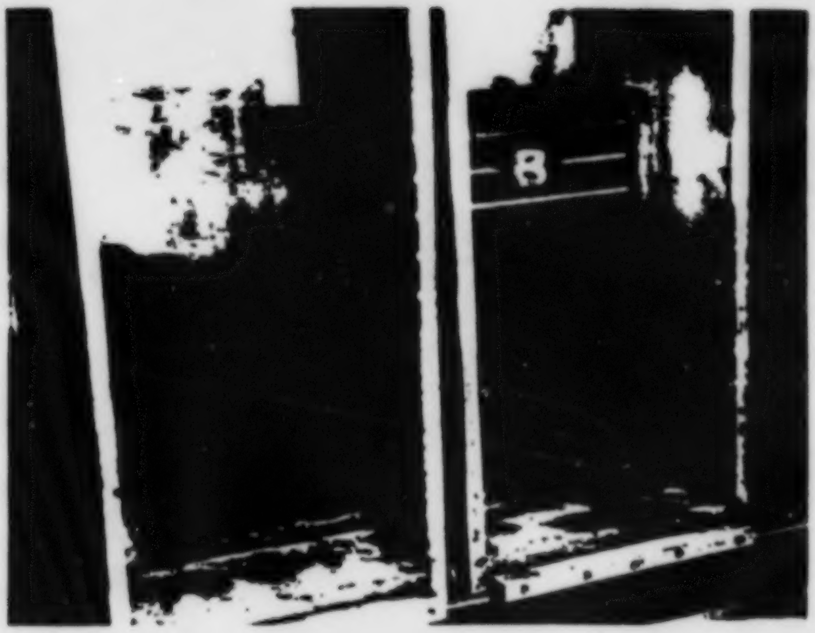
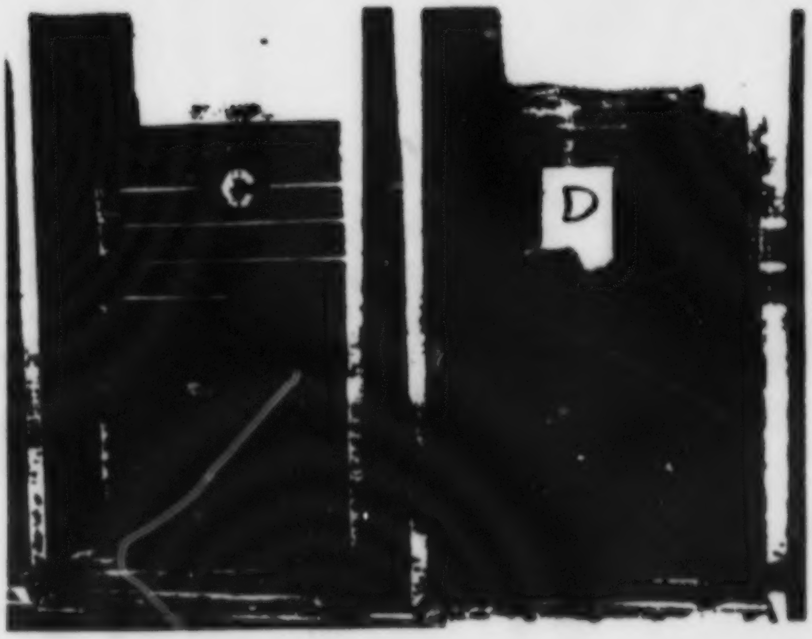


FIGURE 2 CRITICAL HEIGHTS OF BACKFILLS

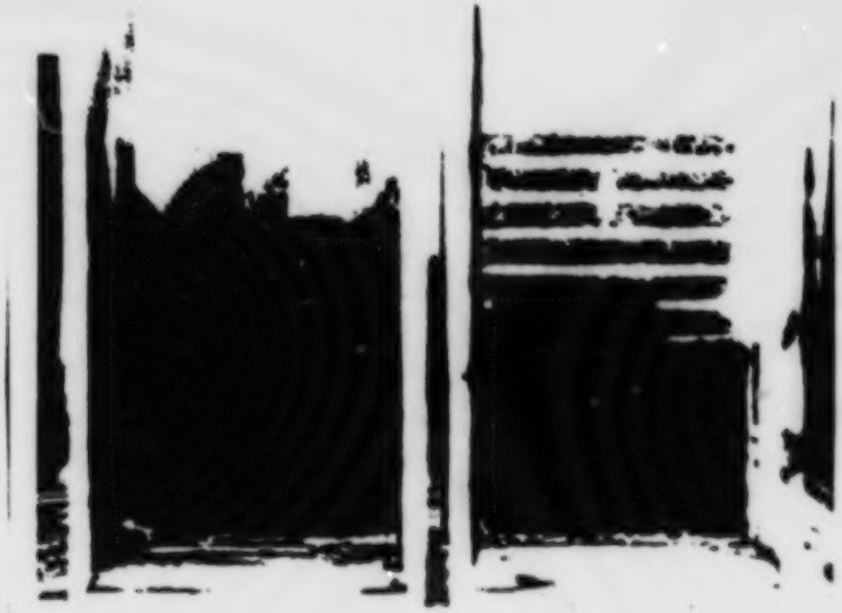


Model A Model B

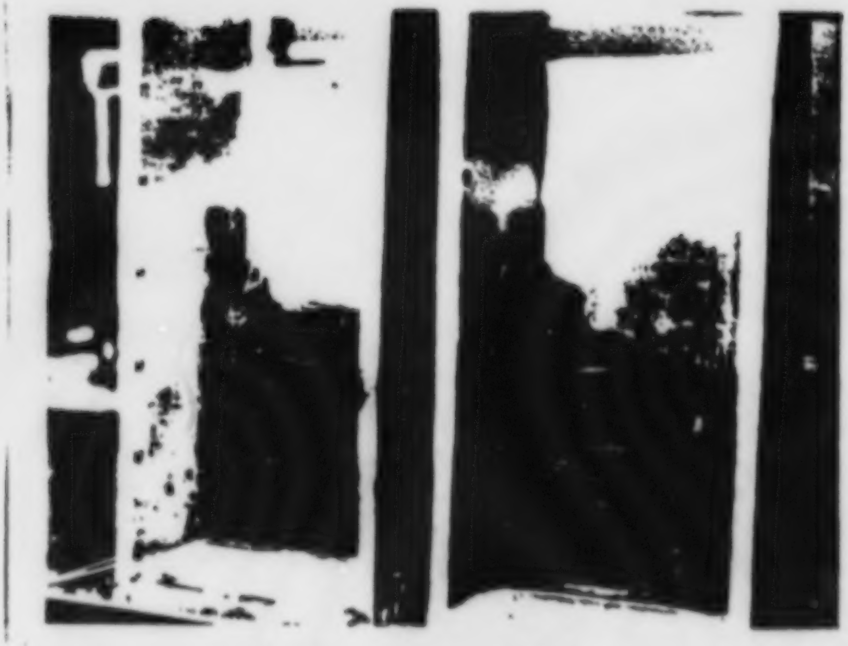


Model C Model D

FIGURE 3 FAILURES IN 50:1 T:C MODELS

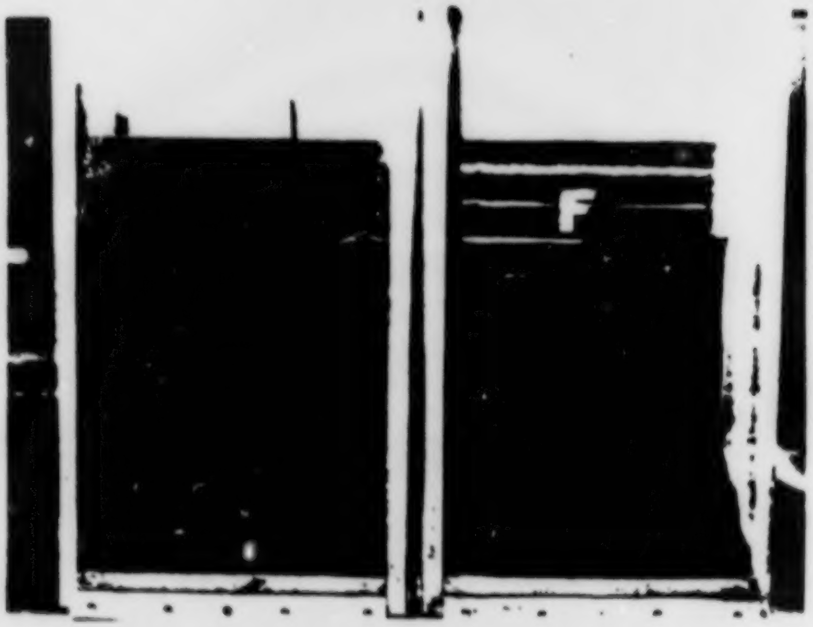


Model J Model I

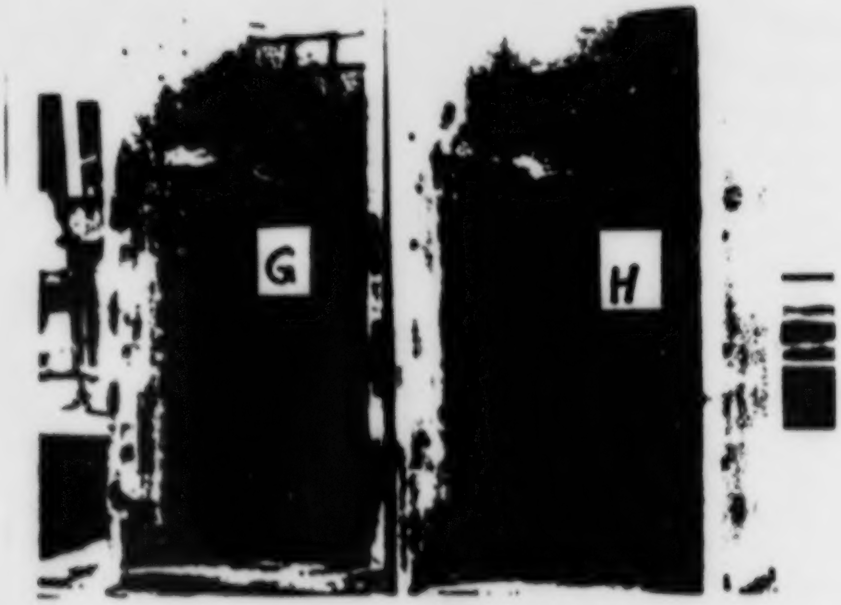


Model K Model L

FIGURE 5 FAILURES IN 33:1 T:C MODELS



Model E Model F



Model G Model H

FIGURE 4 FAILURES IN REINFORCED MODELS

APPENDIX F

CENTRIFUGE MODELLING OF LAYERED FILL

BY

KINGSTON GEORESEARCH

CENTRIFUGE MODEL STUDIES ON LAYERED BACKFILL SYSTEMS

Kingston, Ontario 8 June 1987

INTRODUCTION

The attached report consists of a research paper containing results of 48 centrifuge model tests and discussions and conclusions from analyses of those results, a presentation of photographs of failures in models, suggestions for future research on layered systems and a summary of all the test data. The report is submitted in fulfillment of Phase 1 of a subcontract to carry out centrifugal model studies on backfill alternatives in Ontario. The work reported represents about half the contracted work and much of the remainder is well in progress and should be completed on schedule.

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2 Photographs of model failures not included in the research paper -- 20
3 Suggestions for further research on layered backfill systems --30
4 Appendix A - test results ---------------------------------31

Centrifuge model studies on layered backfill systems

by R.J. Mitchell, Queen's University, Kingston, Canada

ABSTRACT

Plain cemented classified mine tailings backfills have been the subject of considerable industry supported research, due largely to the fact that backfill cement costs can be a significant part of the operating costs in large underground mines **using room and pillar mining methods**. The results of centrifuge model studies reported in this paper show that cement cost savings of up to 50% can be realized for **low fills** by using layered backfill **systems**. Without exceeding 5% Normal Portland cement, **by weight of tailings** sands (20:1 T:C mix), layered systems can extend the stable prototype height of a long fill face from 20 m to 40 m. When the strike length of the exposed fill is less than the fill height and the wall rocks are sufficiently rough to interlock with the fill mass, the stable height of the 20:1 T:C layered system can be in excess of 80 m. **Mass failures in some layered centrifuge models could not be produced at speeds producing stress similitude with 100 m high fills.**

1. INTRODUCTION

Cemented classified tailings are used for backfilling primary stope openings in large underground metalliferous mines throughout the world. Fine sand to silt sized mill tailings and cement are slurried and delivered to underground stope openings by pipeline. The cured fill then supports wall rocks when an adjacent pillar is mined out. The main backfill design criterion are adequate drainage and sufficient cured strength to stand as a vertical wall when exposed by pillar mining sequences. Twenty years ago stope heights seldom exceeded 25 meters and the cemented fill was designed as a free standing wall. The need for higher production, to offset decreasing prices over the past decade, has lead to improvements in drilling and blasting techniques until present stoping heights are approaching 100 m. To cast a plain bulk cemented tailings backfill to this height according to a free standing wall design would be prohibitively expensive. Improved design methods, which consider sand arching, and improved fill design, which consider layered and reinforced systems, are being researched in hope of improving the economics of backfilling. This paper describes the results of 48 centrifuge model tests carried out to determine how layered backfill systems -- systems containing thin strongly cemented layers reinforcing thicker layers of weakly cemented bulk pours -- are expected to perform at a prototype scale. Fibre reinforcement of the thin strong layers is also considered in this study.

2. ~~1.1~~ Centrifuge model preparation and testing

Treated formply was used to construct molds for forming centrifuge models. The classified tailings, having an effective grain size of 0.01 mm, a uniformity coefficient of 13 and a specific gravity of 2.95 were mixed with the appropriate amount of ~~Normal~~ Portland cement at a pulp density of 65% and poured into the molds. Quality control cylinders were also poured for unconfined testing. All models and samples were cured for 28 days prior to testing. The main test series of 48 centrifuge models was composed of six different combinations of tailings to cement ratios (T:C) and layer thicknesses as listed in Table 1. All models were 0.3 m in height, and 0.2 m in strike length: 24 models were 0.15 m in width and 24 models were 0.075 m in width; these are termed wide fills and narrow fills, respectively. Half of the models had smooth side wall conditions during centrifuge testing -- these models were formed to fit into the centrifuge strongbox with a small clearance of about 0.5 mm and developed no significant side wall friction during failure -- and represent the two-dimensional condition of an infinitely long fill. Half of the models had rough side wall conditions, being cast integral with the galvanized steel roughness plates shown on Figure 1. Fast setting plaster was used to ensure a tight fit of the model into the strongbox. This rough wall condition models the case were the stope walls are sufficiently rough to prevent fill boundary movements. In all cases the exposed height to strike length ratio is (H/L) = 1.5.

Based on centrifuge tests on homogeneous 22:1 T:C backfill (4.55% cement), the model scale factor for the layered fill models was designed to be about 100. Thus the 50 mm bulk fill model layer thicknesses are based on a 5 meter prototype fill layer and the prototype strong layers would then be 0.5 and 1 m in thickness. The overall cement usage in the layered models varies from about 3.5 to 5.2% and, thus, the failure heights might not be expected to differ significantly from the 20 m failure height derived from centrifuge testing of the homogeneous 22:1 T:C fill. The strongly cemented layer was reinforced with brass coated shotcrete fibres (0.1 mm diameter by 20 mm long, slightly flattened) in 24 of the models. A mass of 5 g of fibres was used in each layer giving a fibre content for the 10 mm strong layers of 0.85% by weight for the wide fills and 1.7% for the narrow fills. With 5 mm strong layers, these percentages are doubled.

Figure 2 shows a schematic of the Queen's Geotechnical centrifuge. The layered backfill models were fitted into the strongboxes so that the exposed face of each model was perpendicular to the viewing plate and this surface was observed by means of a video camera and digital strobe system while the model was in flight. Video recordings were taped for stop-frame review of the failures. Still photographs were taken after failure of each model and some of these are presented in this paper. Models were conditioned for several minutes at a centrifuge speed of about 60 RPM and then the speed was increased in increments until failure of the models. Incremented acceleration increases were normally in the order of 10 gravities (equivalent to 3 m prototype height) and unfailed models were subjected to a constant acceleration for 5 minutes before the speed was again increased. Experience has shown that there is essentially no time effect in the failure of these brittle cemented sands and failures very often occurred during the slow change in centrifugal acceleration as the centrifuge speed was being increased. The failure speed was noted to the nearest RPM. A time period of five minutes per speed increment was used to ensure that no significant pore water pressures were allowed to build up in the fill. The initial average water content of the models was about 24% of which is below the theoretical saturated value of 27% and a moisture loss of about 7 to 10% was experienced during centrifuge testing. Up to 2.5 mm of settlement (0.8%) was noted in models having smooth walls and weak (40:1 and 100:1 T:C) bulk fills. Moisture loss is mainly due to internal drainage as the simulated gravitational stresses are increased, however, since the rough wall models experience the same moisture loss with a minimal associated settlement. The stronger 30:1 T:C bulk fill also experienced moisture loss with no observable settlements. Strongly cemented layers are relatively low in moisture due to hydration effects. Experience indicates that moisture content and loss of moisture during testing has no significant effect on the stability of these cemented sand backfills.

3. 2 Centrifuge test results

Some basic data from the 48 centrifuge tests and the associated unconfined compression tests are listed in Table 2. It is immediately apparent that the selected model design scale of about 100 proved fairly good for the smooth wall models, the prototype failure heights, λH, averaging around 30 meters. The rough walled models were expected, according to published data and analysis from homogeneous models (see for example, Mitchell et. al. 1982), to fail at upwards to 50% higher scale factors. These layered models, however, did not fail until much higher stress levels. Some models experienced only facial sloughing failures at centrifuge speeds of close to 300 RPM (the normal limit to which the machine is operated). Indeed, these rough wall models were only marginally within the testing capabilities of this centrifuge. Figure 3 shows the narrow rough wall models 3 and 4 after five minutes at 325 RPM. The white lines are spray painted onto the models using a template to improve the in-flight visual effects. A small amount of base spalling is the only sign of distress in these models at a prototype height of 90 meters. The wide rough walled models 21 and 22 showed similar strengths and were not failed at 290 RPM. At the scale factor (a = 252 g for

290 RMP) the 50 mm of 30:1 bulk fill has a prototype equivalent of 12.5 m.

Figure 4 shows the failures in the narrow smooth wall models 3 and 4. Shear failure through the bulk fill and two or three strong layers is typical of the narrow smooth wall models. The wide smooth wall models with 50 mm layers of low bulk cement content (40:1 and 100:1 T:C) also failed by shear through the base portion, as typified by models 13 and 14 shown, after failure, on Figure 5. The wide smooth wall 30:1 T:C bulk fills failed by propagation of a failure plane (and possible tension crack) through to the fill surface as shown by models 15 and 16 on Figure 6. Occasionally the fibre reinforced layers prevented a rapid shear/tension failure and testing of model 19 was stopped to document the major base shear/caving failure shown on Figure 7. The wide smooth wall models with the 30 mm layer spacings (models 23 and 24) and 100:1 T:C bulk fill failed by base sloughing followed by mass failure. In the case of model 23 roughly 50% of the face actually failed as shown on Figure 8. This could have resulted due to an uneven distribution of the fibre reinforcements.

The rough wall models generally failed by sloughing (caving of material from between the strong layers) from the base upwards as typified by models 9 and 10 on Figure 9 and 15 and 16 on Figure 10. The rough wall models with fibre reinforced layers would not exhibit mass failure within the limits of the testing speeds used but the plane layer models usually exhibited mass failure extending back about 50 mm into the backfill once the caving mechanism had progressed about half way up the model. These failures did not appear to be influenced at all by the model width. Figure 11 shows a model following manual excavation of the bulk fill between the layers and is included to demonstrate that thin strong model layers were formed within acceptable tolerance.

4. ~~3~~ Analyses of the model data

Before analyzing the centrifuge data, the quality control test results were plotted, as shown on Figure 12 to delineate any major anomolies. Smooth wall models 7 and 8 were found to have abnormally weak layers and rough wall models 19 and 20 were found to have relatively strong layers. The rest of the data is reasonably consistent with statistically expected variations. It is possible that some mixing error occurred in the anomolous models but the abnormal strength differences are more likely a result of an inclusion of excess fines in the bag of tailings used for mixing models 7 and 8. These experimental problems are eliminated, in comparing the model data, by correcting all of the T:C designations according to the actual strengths as measured in the control samples. Following this strength correction, the prototype failure heights are plotted against the total cement usage (as a percent of total dry tailings weight) on Figure 13. The following specific conclusions are obtained from the data on Figure 13:

1. All layered fill systems are superior to plain bulk pour having the same overall cement usage.

2. Fills confined by rough walls are much more stable than similar fills confined by smooth walls.

3. The stable height of fill generally increases with the overall cement usage. Although there is considerable variation in the data, there is no

clear indication that there is a preferential bulk pour cement content nor strong layer thickness.

4. All smooth wall models failing below 32 m prototype height are either 100:1 bulk T:C models or models with thin strong layers. All rough wall models failing below 75 m prototype height are in the same categories. Thus, the thicker strong layers and higher bulk T:C ratios appear to be superior.

5. If prototype uncemented bulk poured tailings were to drain and densify under self-weight stresses so as to have shearing resistance equivalent to laboratory 100:1 T:C mixes, strong layers poured at regular spacings in uncemented bulk pours would be superior particularly in the smooth wall cases (the arrows show the reduction in overall cement usage for models where uncemented tailings could be substituted for the 100:1 T:C bulk material).

Table 3 contains the strength corrected model descriptions and a variety of calculated ratios. The most significant ratios, those of the critical heights of rough to smooth wall models, show that rough walls increase the stable height by an average factor of 2.3. This result is most surprising because previous analytical results which considered shear on rough walls in plain fills (Mitchell, et. al. 1982) indicate only a marginal 25% increase due to rough walls at the height to strike length ratio of $H/L = 1.5$ used in these models. It has been noted, however, that video and still photos show that the rough wall condition in layered fills alters the failure mechanism from a single shear plane to a mechanism involving a curved shear plane which exists in the surface of the sample. Indeed, some failures in rough wall models were of an interlayer sloughing type and the fill mass could not be brought down within the limits of the centrifuge. These observations indicate that the combination of wall roughness and strong layers prevents lateral movement (which may have tended to cause tensile cracking of the strong layers in the smooth wall models) and hence prevents the development of a normal shear plane. Figure 4 contains an approximate analysis of a very steep shear plane after base sloughing has developed. It is shown that the critical height increases as the failure plane is forced to a steeper location. Model evidence indicates that the failure plane goes back about 20% of the model height, giving $\alpha = 78°$. For this value of γ, the critical height (Hc) is given as $9.6\ \tau/\gamma$ or $0.5\ \tau$ m for $\gamma = 19\ KN/m^3$. A 5/30, 1/10 model would have an average UCS of about 260 KPa or $\tau = 130$ KPa, giving $Hc = 65$ m. This value is a lot closer to the 77 m derived from models 15 and 16 than the value of 27 m given by the normal upper bound equation $Hc = 2\ (UCS)/\gamma$. It is clear that centrifuge model studies are useful in confirming kinematic restrictions on mass movements. It is also clear that wall roughness is a matter deserving further research.

It is interesting to note from Table 3 that the full width had no effect on failure with rough walls (values of 0.72 to 0.94 in this Table are more a measure of the degree of sloughing achieved at the ultimate test speed than of actual stability). With the smooth wall, however, the wide fills had about 25% higher critical heights as expected from the usual analysis (see Mitchell, 1983, page 232).

Finally, Table 3 shows that fibre reinforcement of the strong layers provided only an average 15% increase in critical height. Since 5 g of shotcrete fibres were used in each reinforced layer, the concentrations were about 9 kg/tonne in the wide model layers and 18 kg/tonne in the narrow model layers for overall concentrations of 1.5 and 3 kg/tonne of backfill, respectively, in the models having bulk pour thicknesses of 5 times the layer thicknesses. At two dollars per kg of fibres, the lower concentration represents a cost of $1.5/tonne of backfill and it is concluded that the cost-benefit ratio is too high to consider using this type of inclusion. It should be noted, however, that the rough to smooth wall critical height ratio varies from 1.84 for the wide fibre reinforced models to 2.73 for the narrow fibre reinforced models. This indicates a significant increase in stability due to the increase in fibre concentration. If less expensive types of fibres become available, their use might become economically viable.

5 ~~4~~ Conclusions

Centrifuge model test data presented in this paper clearly show that layered backfill systems are vastly superior to plain cemented bulk pours, producing fills that stand to twice the height with smooth walls and up to four times the height with rough walls. Several different combinations of tailings to cement ratios and layer thicknesses were tested and, in general, thicker strong layers and stronger bulk layers were found superior for the same overall cement usage. More detailed conclusions are contained in the paper. Although 48 centrifuge model tests were carried out, no attempt has been made to optimize the strong layer spacings. Shotcrete fibres were used as reinforcing inclusion in the strong layers of 24 models and it is concluded that this type of reinforcement is not cost-effective. The present research effort to develop better (safer and less expensive) backfills for large underground mining operations is a relatively recent development. This paper presents the first centrifuge model tests on layered backfill systems and the results indicate that model studies on layered and reinforced backfill systems should be continued.

Acknowledgements

Queen's geotechnical centrifuge was constructed in 1985 from funds provided by the Natural Sciences and Engineering Research Council, Falconbridge Ltd. and Westmin Resources Ltd. The research reported in this paper was contracted with Falconbridge Ltd. Mr. Jim Roettger, Research Assistant, helped to prepare and test the models.

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Strong layers		bulk layers	
thickness, mm	T:C	thickness, mm	T:C
1	8:1	5	40:1
1	10:1	5	30:1
1	6:1	5	100:1
0.5	5:1	5.5	40:1
0.5	8:1	5.5	30:1
1	6:1	3	100:1

Table 1 - Layered Backfill Models

Model No.		Nominal model description (see note)	UCS control, KPa		Centrifuge model failure data			
			Smooth walls mass/layer	Rough walls mass/layer	Smooth walls		Rough walls	
					RPM	λH m	RPM	λH m
1	N	5/40, 1/8 F	1160/90	1360/80	150	20	310	87
2	A	5/40, 1/8 P			150	20	290	76
3	R	5/30, 1/10 F	905/135	1110/165	202	37	325	95
4	R	5/30, 1/10 P			188	32	310	87
5	O	5/100, 1/6 F	1970/45	2460/45	170	26	245	54
6	W	5/100, 1/6 P			145	19	220	44
7		5.5/40, 0.5/5 F	1260/85	3260/75	100	9	290	76
8	M	5.5/40, 0.5/5 P			100	9	260	61
9	O	5.5/30, 0.5/8 F	1320/140	1260/115	184	30	305	84
10	D	5.5/30, 0.5/8 P			184	30	295	78
11	E	3/100, 1/6 F	2010/50	2030/50	185	31	310	87
12	L	3/100, 1/6 P			185	31	280	71
13	S	5/40, 1/8 F	1380/90	1300/75	203	37	295	78
14		5/40, 1/8 P			184	30	295	78
15	W	5/30, 1/10 F	920/130	950/115	206	38	295	78
16	I	5/30, 1/10 P			200	36	290	76
17	D	5/100, 1/6 F	1970/50	2025/35	175	28	245	54
18	E	5/100, 1/6 P			133	16	235	50
19		5.5/40, 0.5/5 F	2910/85	4350/115	210	40	285	73
20	M	5.5/40, 0.5/5 P			170	26	260	61
21	O	5.5/30, 0.5/8 F	1345/140	1530/145	215	42	290	76
22	D	5.5/30, 0.5/8 P			185	31	290	76
23	E	3/100, 1/6 F	2070/50	1910/40	213	41	275	68
24	L S	3/100, 1/6 P			195	34	265	63

NOTE: The model description 5/40, 1/8 F means 5 cm of 40:1 T:C with an overlying layer of 1 cm of 8:1 T:C containing fibres. The letter P indicates plain layers (no fibre reinforcement). Samples were mixed and poured in pairs.

Table 2 - Basic Data From Centrifuge Tests

			Ratio R/S		Ratio F/P		Ratio W/N [2]	
Corrected model description [1]	Model depth mm	Model Nos.	Fibre reinf.	Plain layers	Smooth wall	Rough wall	F/P Smooth wall	F/P Rough wall
5/40, 1/8.7	75	1,2	3.71	3.24	1.0	1.14	1.55	1.01
5/40, 1/7.8	150	13,14	2.23	2.76	1.22	1.0	1.26	1.07
5/29, 1/10.2	75	3,4	2.08	2.20	1.16	1.09	1.01	0.96
5/30, 1/10	150	15,16	1.99	2.04	1.06	1.03	1.11	1.02
5/100, 1/6.1	75	5,6	1.66	1.85	1.15	1.23	1.08	1.21
5/77, 1/6.2	150	17,18	1.88	3.04	1.64	1.08	0.84	1.38
5.5/42, 0.5/8.3	75	7,8	3.26	2.62	1.0	1.25	1.92	0.72
5.5/42, 0.5/5.1	150	19,20	1.22	1.57	1.45	1.20	1.25	0.75
5.5/30, 0.5/8	75	9,10	2.93	2.72	1.0	1.08	1.37	0.75
5.5/30, 0.5/8	150	21,22	1.59	2.16	1.29	1.0	1.01	0.80
3/77, 1/6	75	11,12	2.78	2.27	1.0	1.23	1.28	0.83
3/77, 1/6	150	23,24	2.17	2.00	1.19	1.08	1.06	0.94
	averages		2.29	2.37	1.18	1.12	1.23	0.95

---

1. Tailings to cement ratios are corrected with reference to the control cylinder strength. Only models 7,8 (low strength layers) and models 19,20 (very high strength layers) are significantly effected.

2. The quoted ratios are R (rough) to S (smooth), F (fibre) to P (plain) and W (wide) to N (narrow).

Table 3 - Ratios of Critical Heights

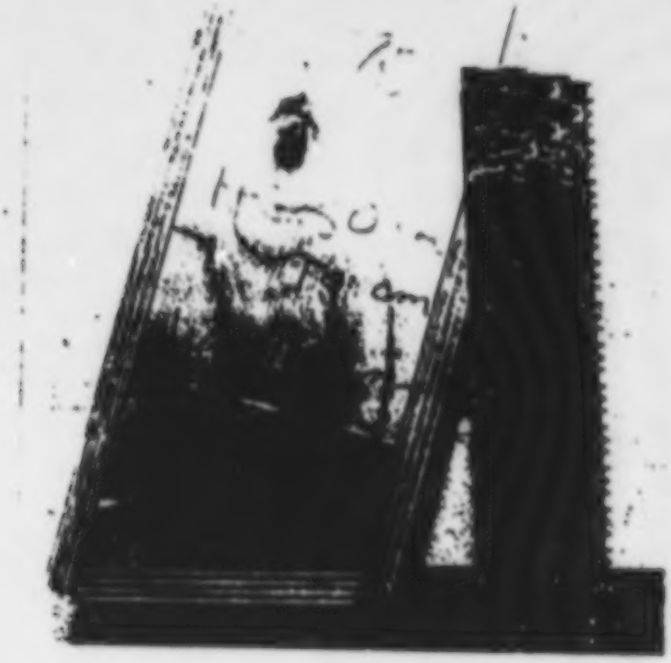


FIGURE 1 STEEL WALL ROUGHNESS

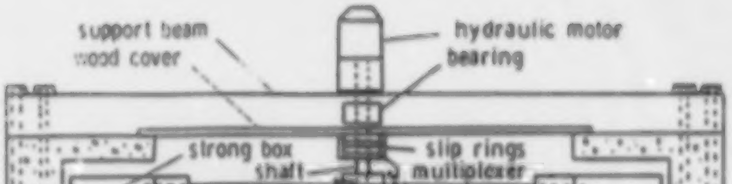

support beam
wood cover
hydraulic motor
bearing
strong box
shaft
slip rings
multiplexer
radius arm
terminal bar
bearing
3m
strong box to beam connection pin
remote controlled multipexing switch
reinforced concrete housing
strong box in access opening

FIGURE 2 QUEEN'S GEOTECHNICAL CENTRIFUGE

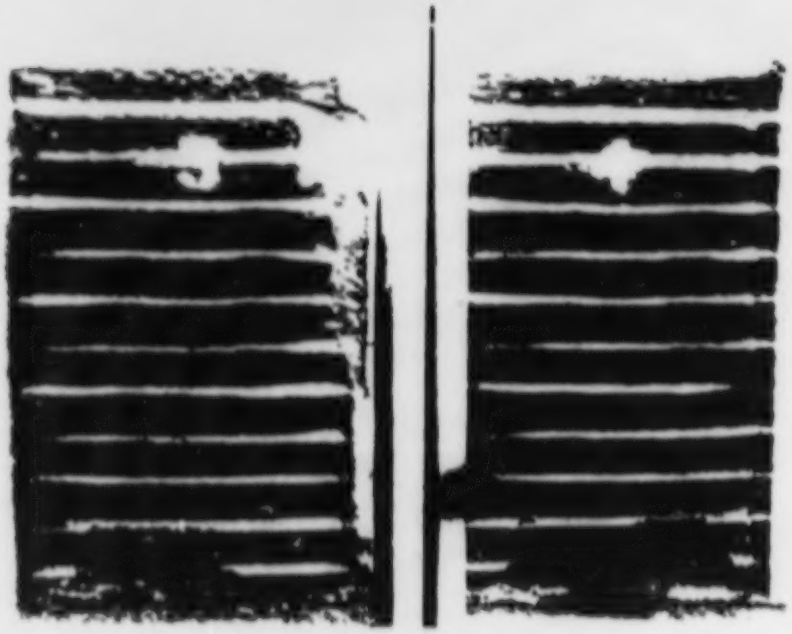


FIGURE 3 ROUGH WALL MODELS 3 & 4 AFTER 315 GRAVITIES

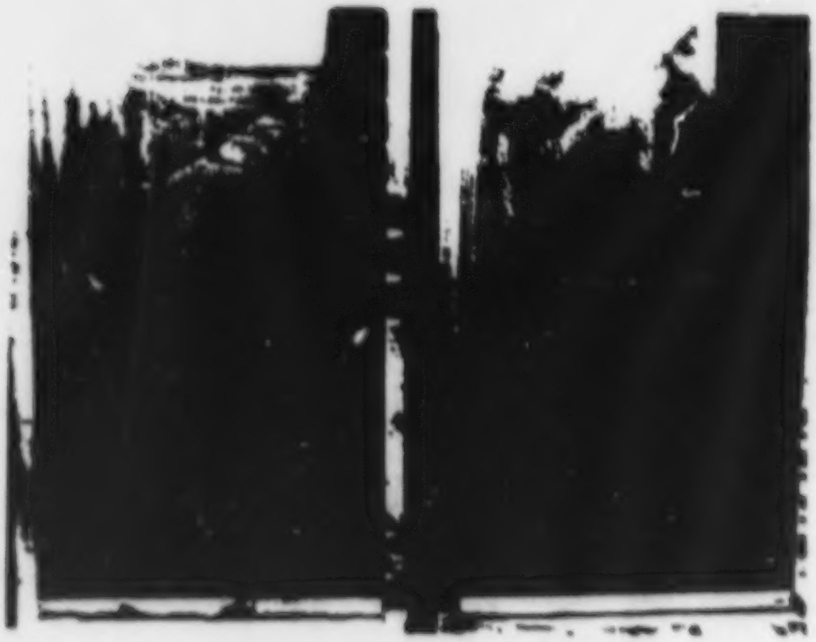


FIGURE 4 SMOOTH WALL MODELS 3 & 4 AFTER FAILURE

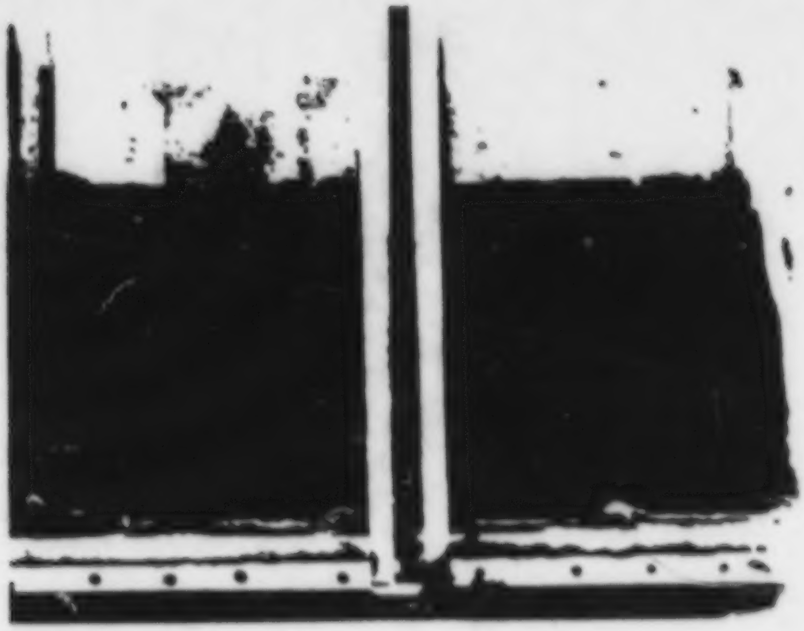


FIGURE 5 SMOOTH WALL MODELS 13 & 14 AFTER FAILURE

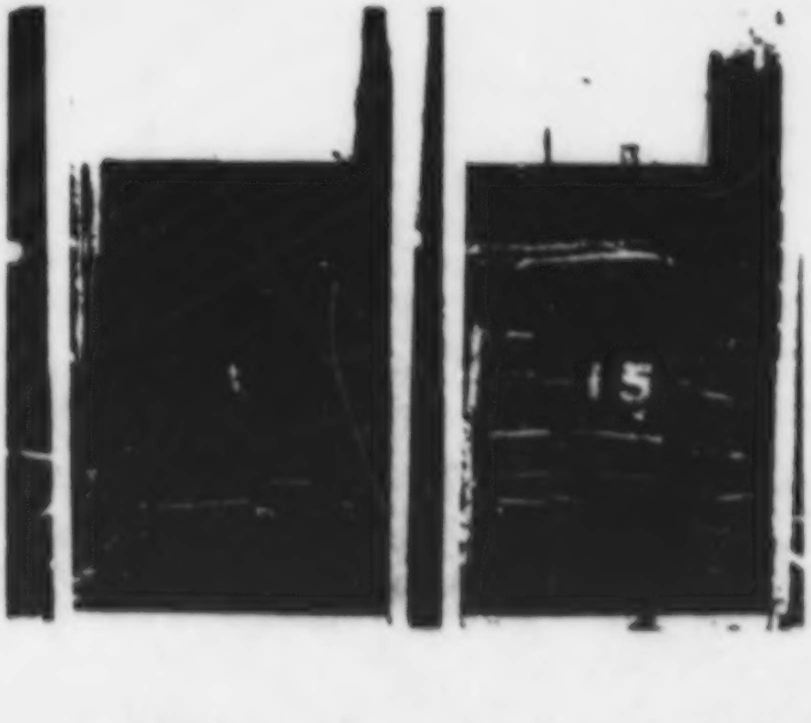


FIGURE 6 SMOOTH WALL MODELS 15 & 16 AFTER FAILURE

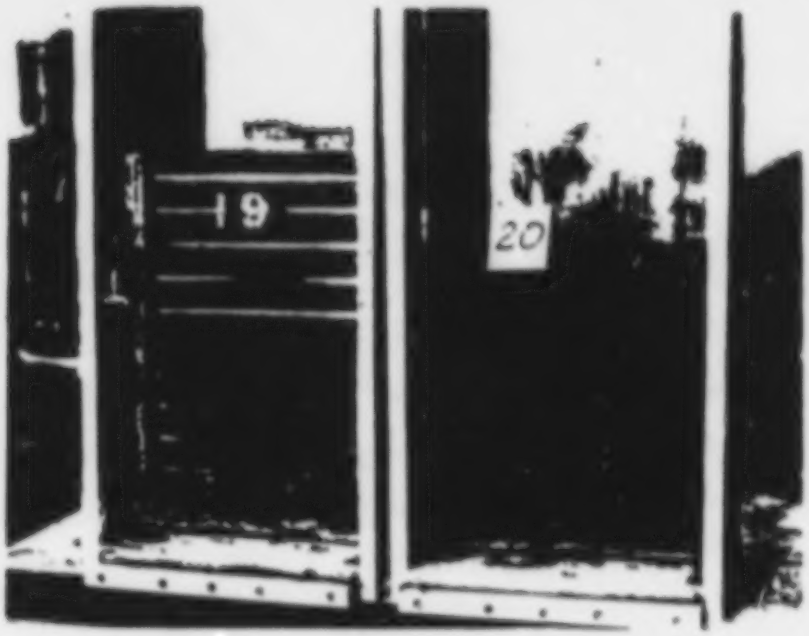


FIGURE 7 SMOOTH WALL MODELS 19 & 20 AFTER FAILURE

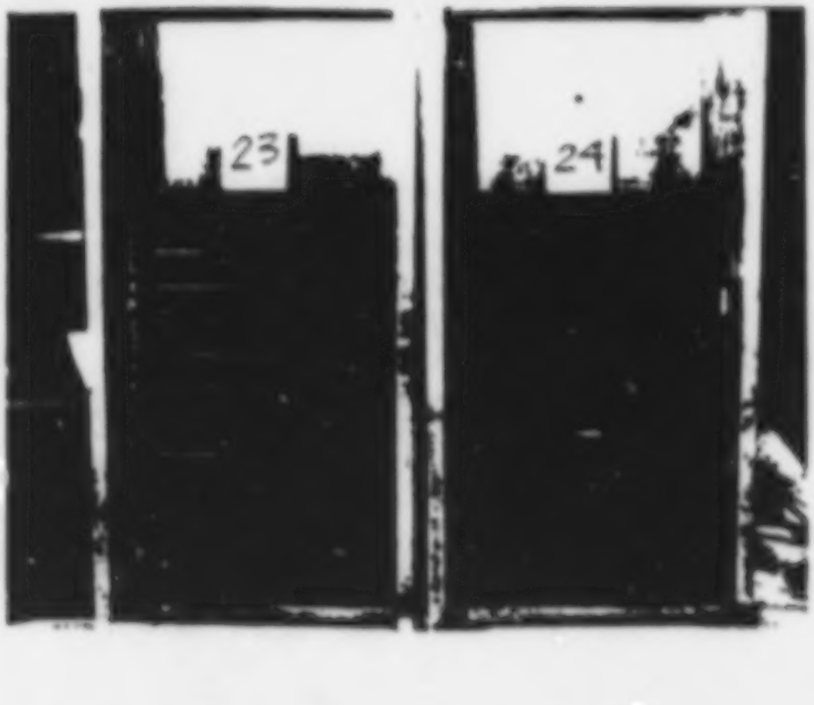


FIGURE 8 SMOOTH WALL MODELS 23 & 24 AFTER FAILURE

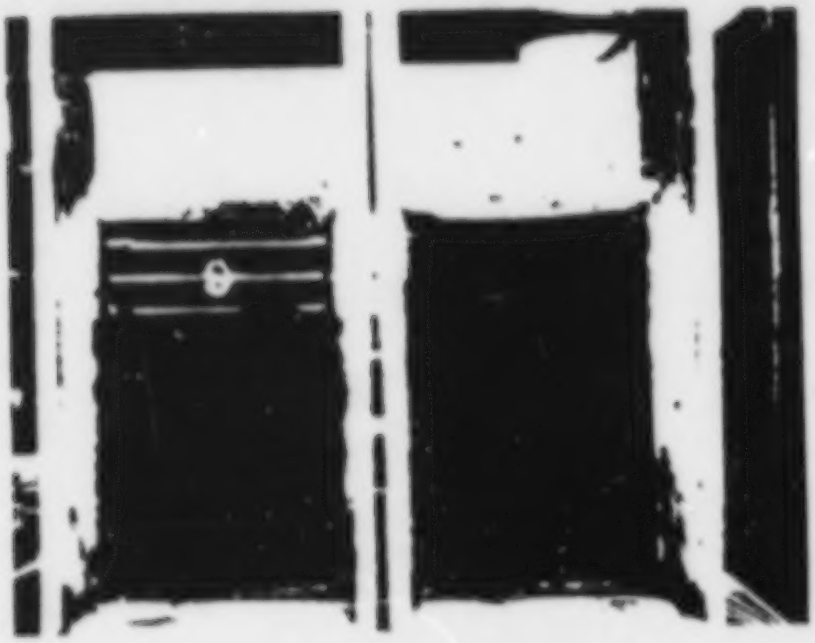


FIGURE 9 ROUGH WALL MODELS 9 & 10 AFTER 280 GRAVITIES

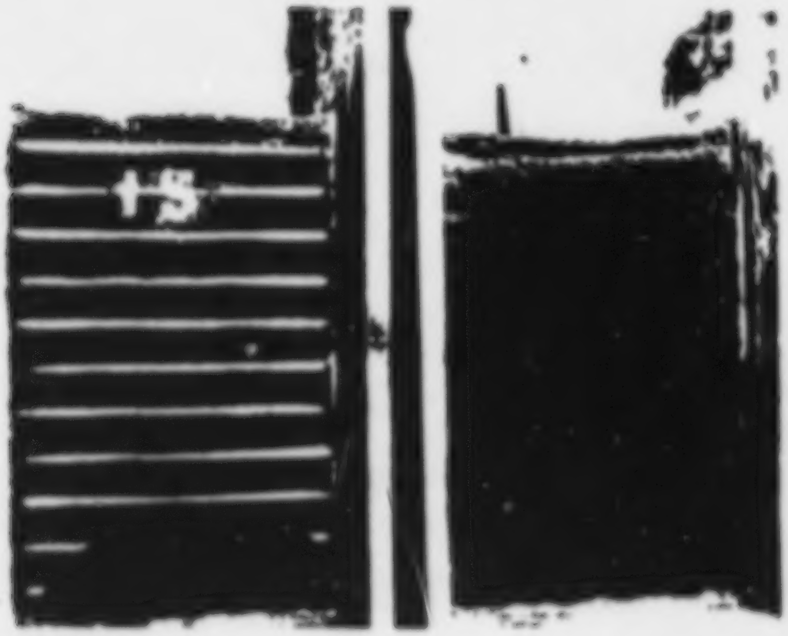


FIGURE 10 ROUGH WALL MODELS 15 & 16 AFTER 260 GRAVITIES

FIGURE 11 THIN LAYERS IN MODEL 18

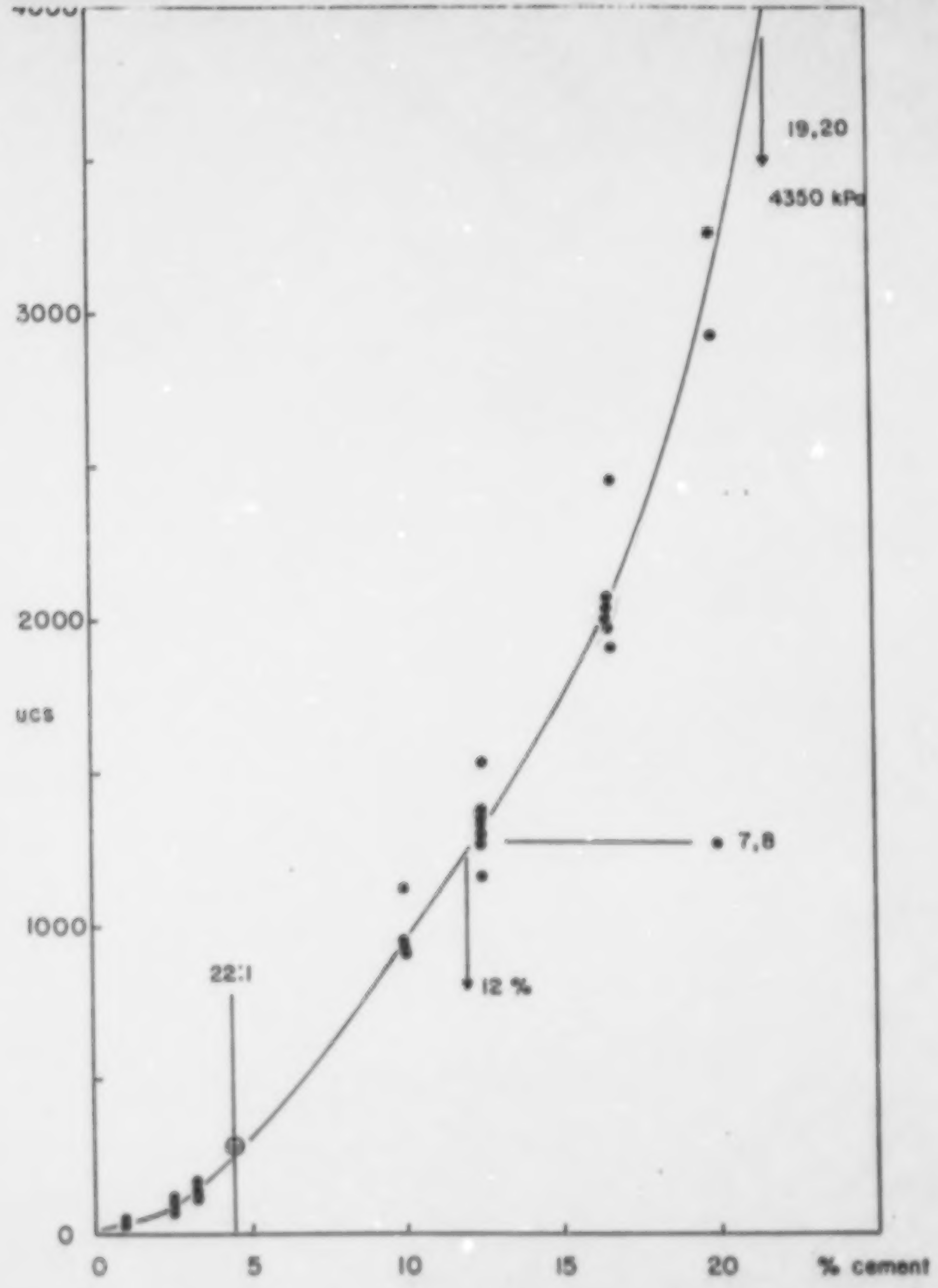


FIGURE 12 UNCONFINED COMPRESSIVE STRENGTH

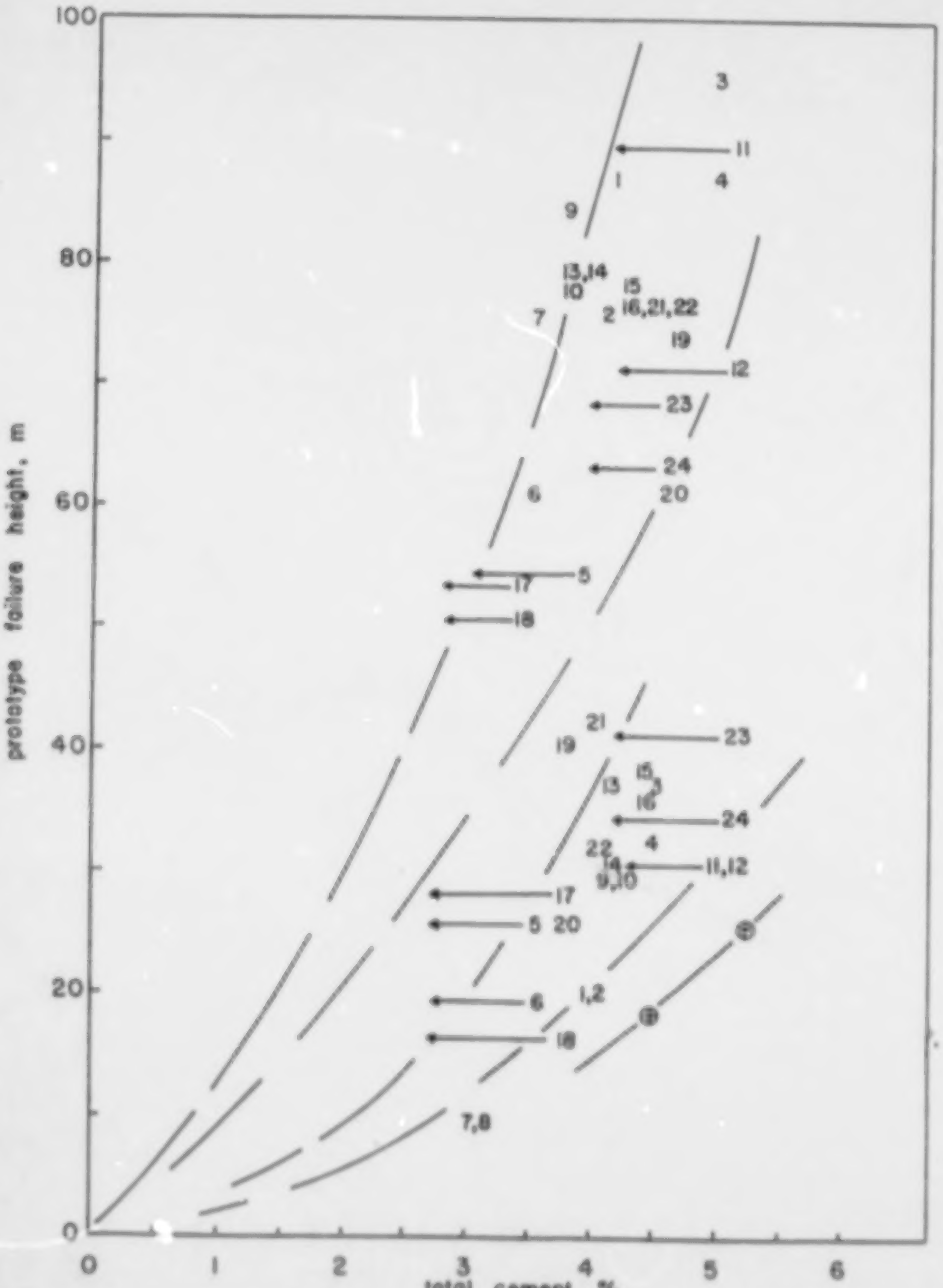
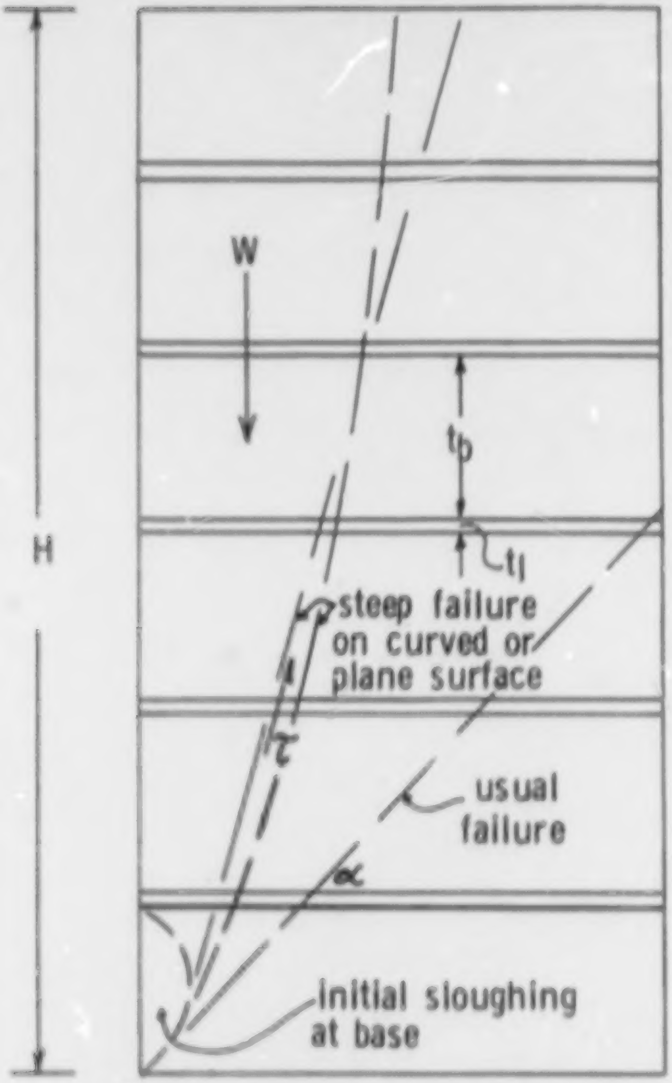


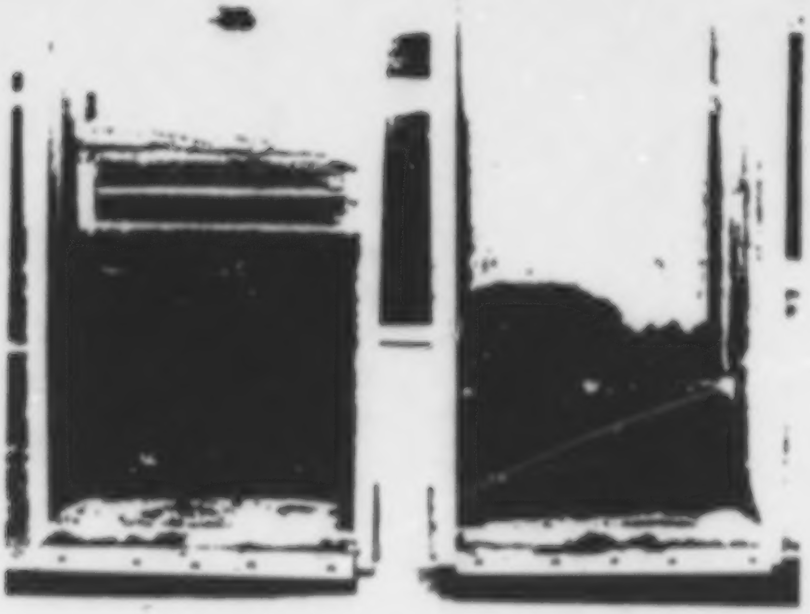
FIGURE 13 PROTOTYPE FAILURE HEIGHTS


W
H
t_b
t_l
steep failure on curved or plane surface
τ
usual failure
α
initial sloughing at base

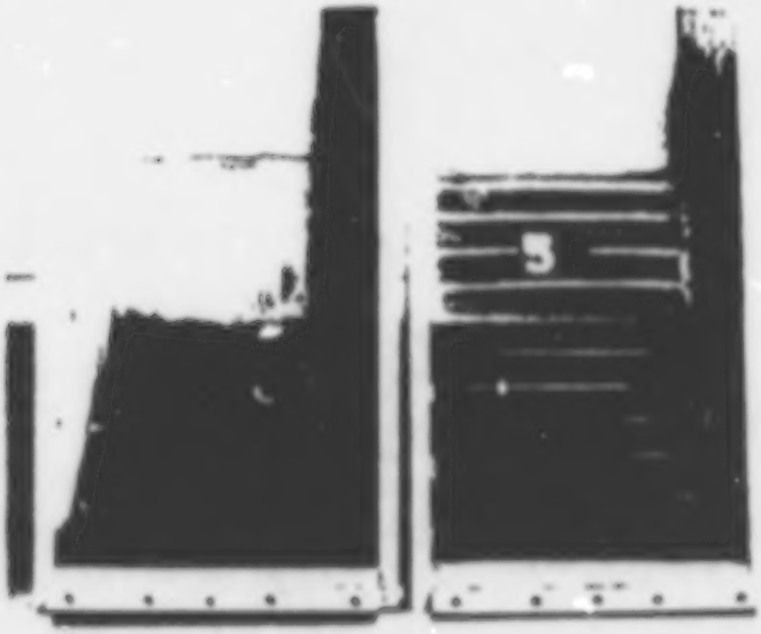
$$\tau = [(t_l\,(UCS)_l + t_b\,(UCS)_b)]/2(t_l + t_b)$$

FIGURE 14 FAILURE OF LAYERED FILLS ON STEEP SLIP SURFACES

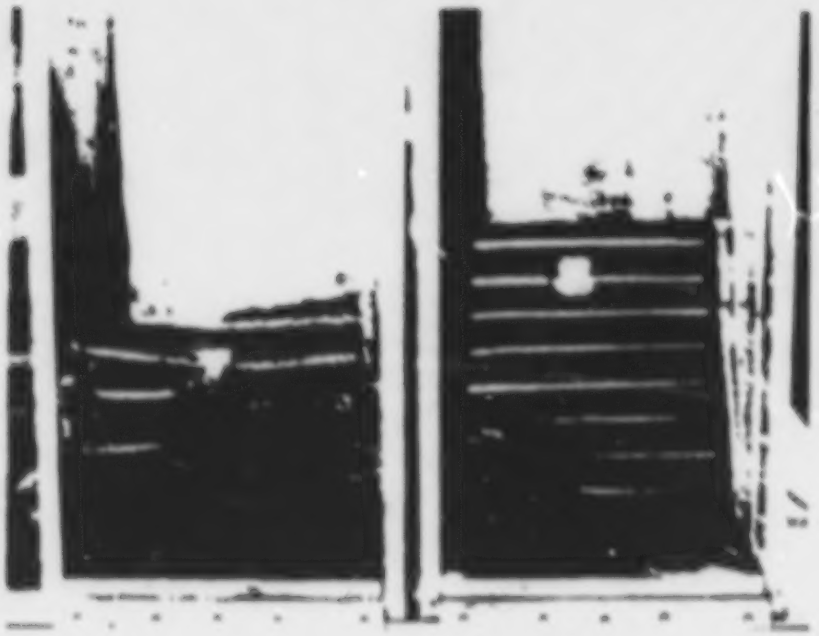
PHOTOGRAPHS OF MODEL FAILURES



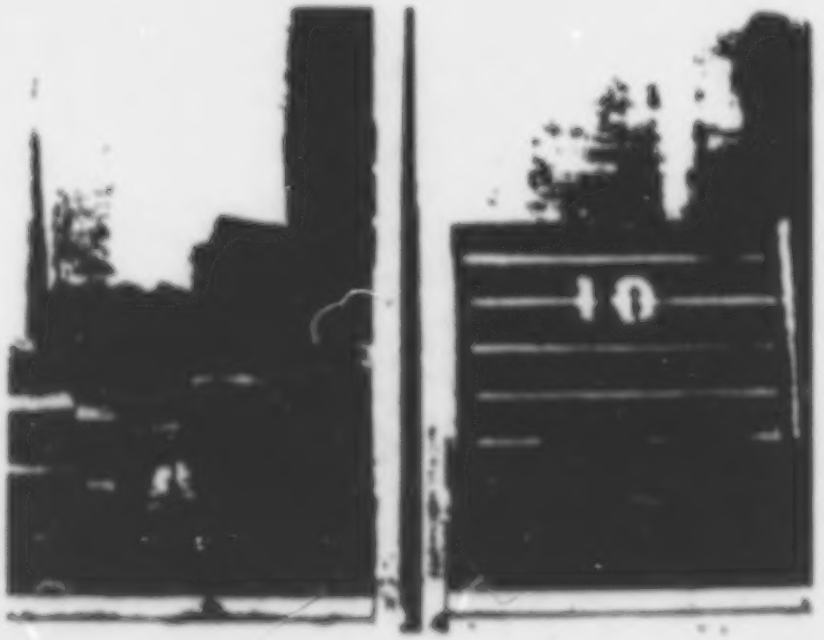
SMOOTH WALL MODELS 1 & 2 AT FAILURE



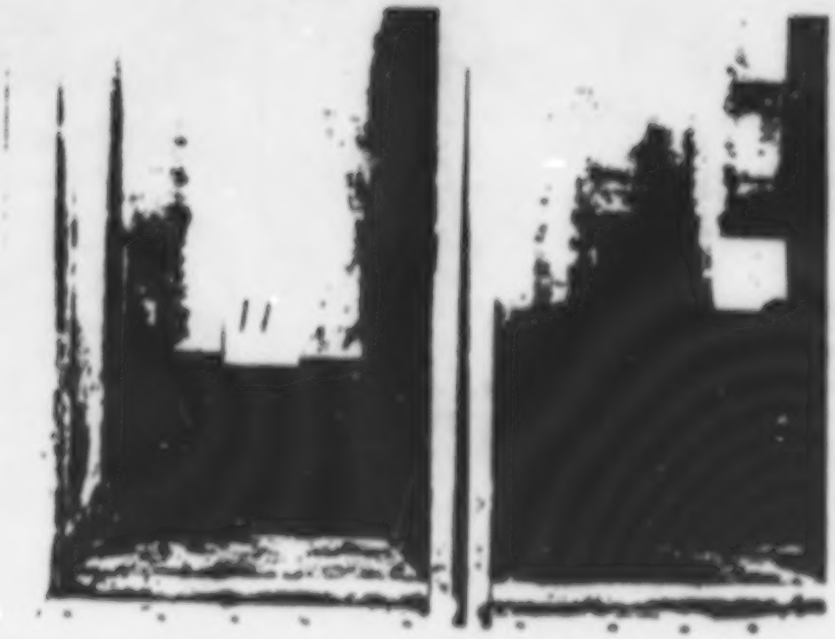
SMOOTH WALL MODELS 5 & 6 AT FAILURE



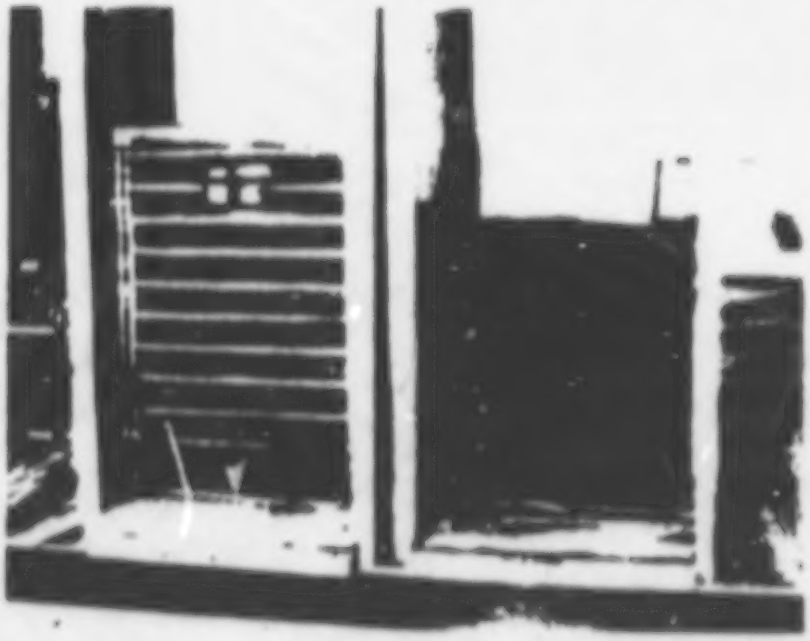
SMOOTH WALL MODELS 7 & 8 AT FAILURE



SMOOTH WALL MODELS 9 & 10 AT FAILURE

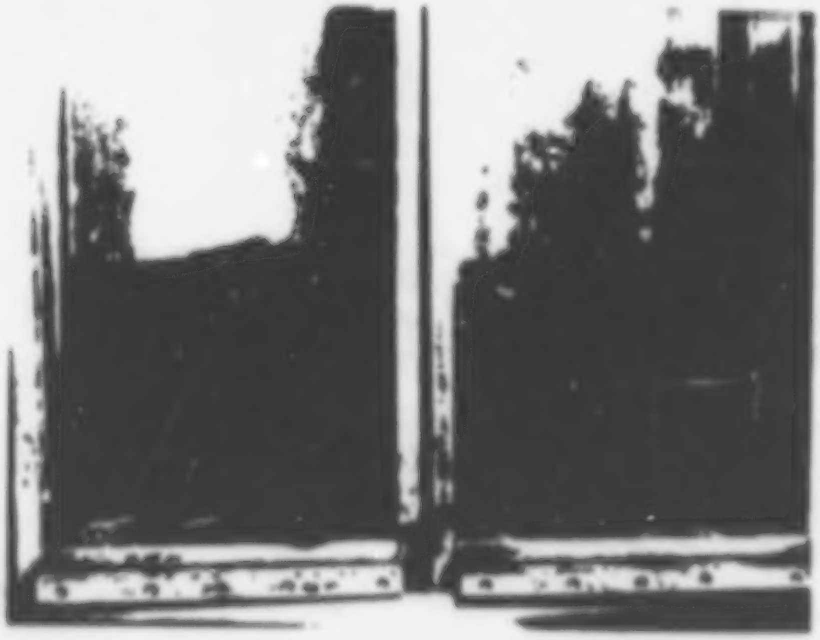


SMOOTH WALL MODELS 11 & 12 AT FAILURE

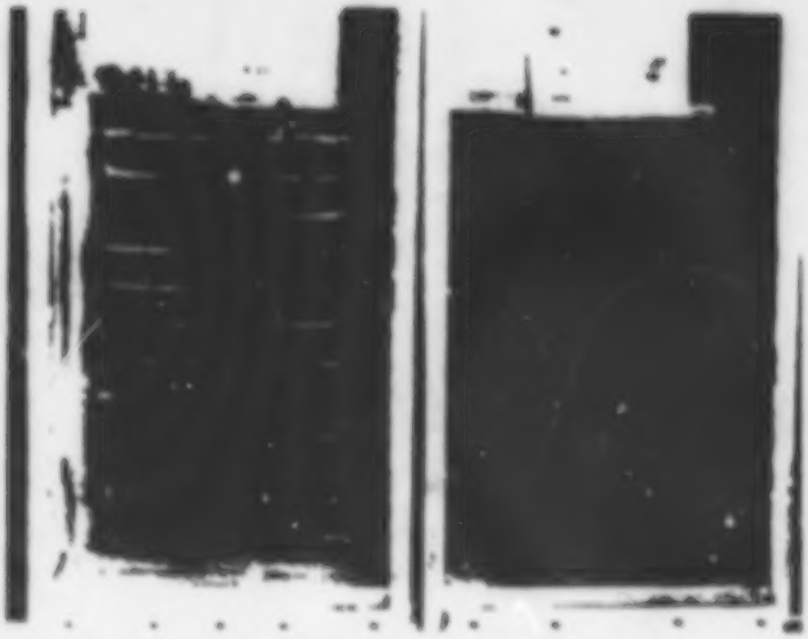


SMOOTH WALL MODELS 17 & 18 at 28m and failure

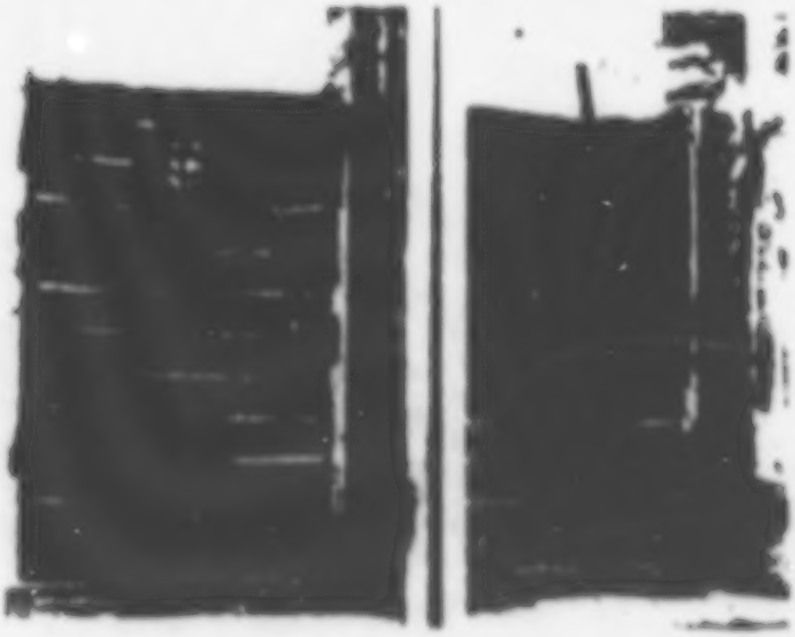
SMOOTH WALL MODELS 21 & 22 at failure and 42m
(note:models misnumbered-22 contains fibres)



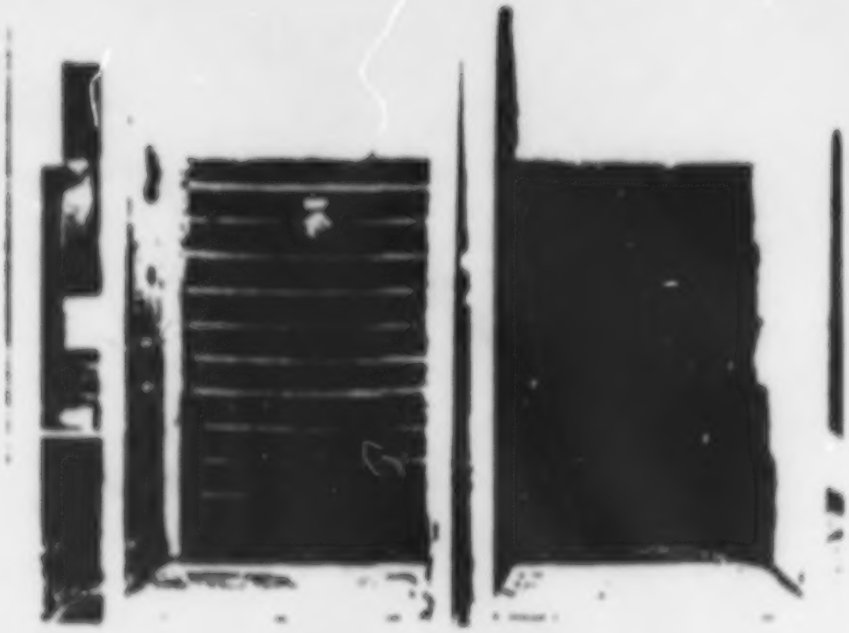
SMOOTH WALL MODELS 23 & 24 AFTER FAILURE



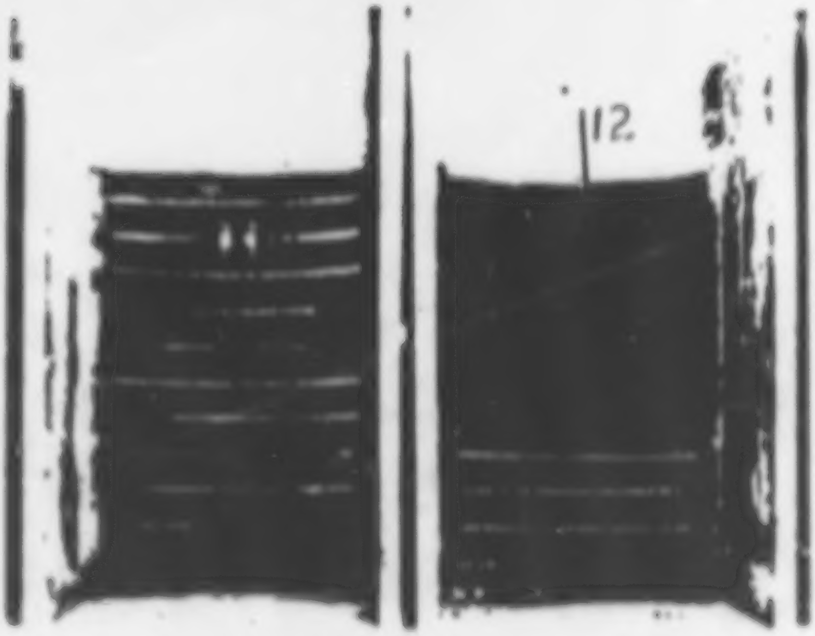
ROUGH WALL MODELS 1 & 2 AT 87 and 76 m



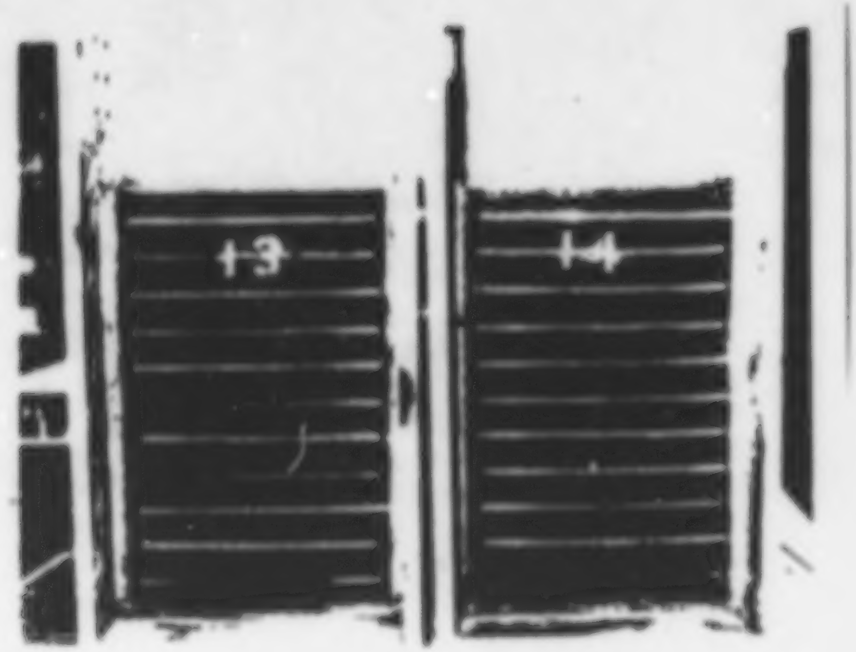
ROUGH WALL MODELS 5 & 6 AT 54m and 44m



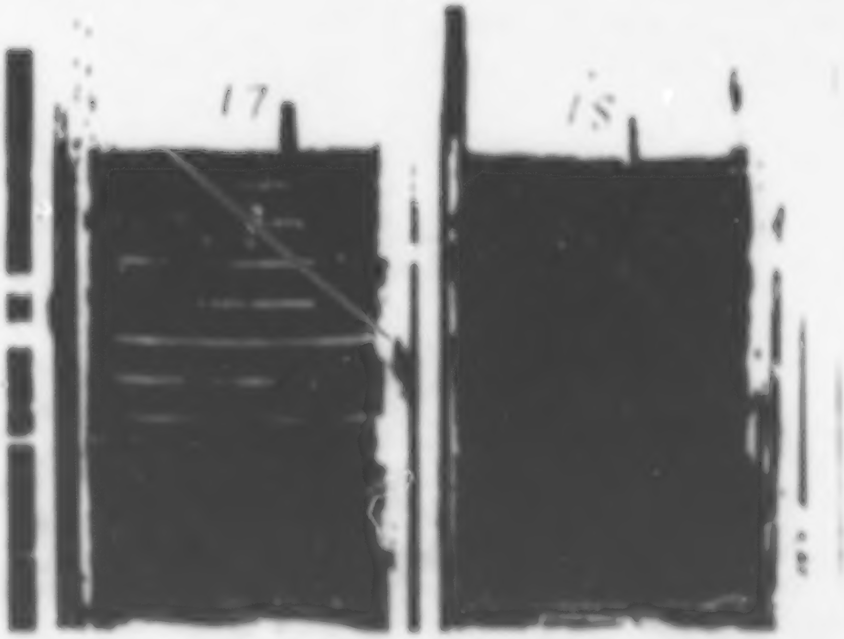
ROUGH WALL MODELS 7 & 8 AT 76m and 61m



ROUGH WALL MODELS 11 & 12 AT 87m and 71m

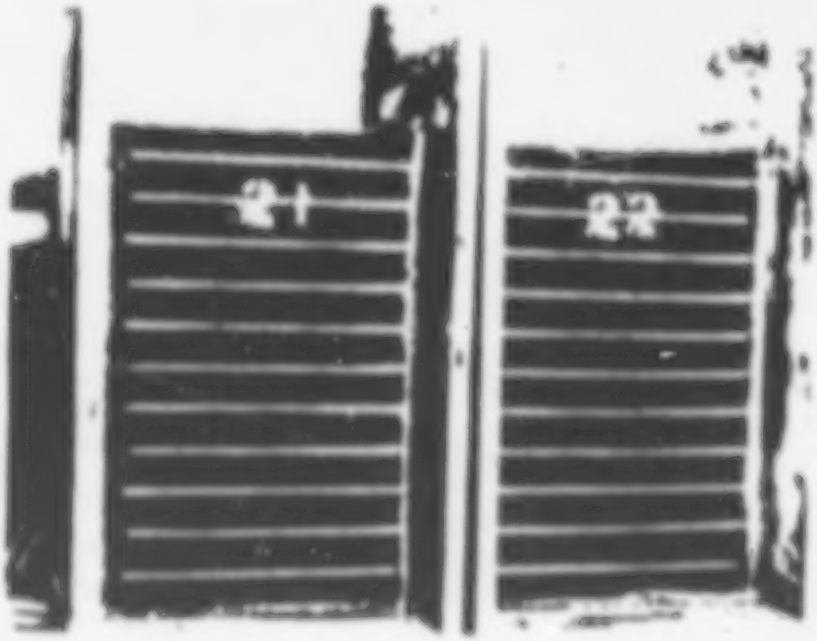


ROUGH WALL MODELS 13 & 14 AT 78m PROTOTYPE HT.



ROUGH WALL MODELS 17 & 18 AT 54m and 50m

ROUGH WALL MODELS 19 & 20 AT 73m and 61m



ROUGH WALL MODELS 21 & 22 AT 76m PROTOTYPE HT.

ROUGH WALL MODELS 23 & 24 AT EQUIVALENT PROTOTYPE HEIGHTS OF 68 and 63m

SUGGESTIONS FOR FURTHER RESAEARCH ON LAYERED BACKFILLS

The model studies reported in this research were carried out to see if layered systems, in general, improved the stability of cemented tailings backfills. No theoretical nor physical model results were developed which could result in optimization of the layer design thicknesses and cement contents for different stope conditions. Because layering has been proven to be quite effective, it is suggested that optimization studies be conducted.

The two limiting conditions of plane smooth walls and extremely rough interlocking stope walls were modelled. Intermediate wall roughness conditions that might be appropriate for real mine rock types should be modelled using the optimized designs for layered backfills.

Considerable savings could be realized, both in cement costs and in operational costs, if the weak layers could be uncemented fill. There is some indications from previous research that 100:1 T:C is not any stronger than uncemented fill that has been allowed to consolidate (in the soil mechanics sense) under the self-weight of overlying fill.[1] Several models should be prepared by using centrifugal accelerations to simulate the self-weight consolidation prior to curing of the model (i.e.simulation of the entire pour process) with optimized 6:1 T:C layers placed in uncemented fill.

We would be pleased to be involved in continued research on this most interesting topic of backfill engineering.

[1] It was necessary to use some cement in the weak layers for the present models because of handling from molds to strongbox.

APPENDIX G

CENTRIFUGE MODELLING OF SLOPED AND DENSE WALL FILL

BY

KINGSTON GEORESEARCH

PRELIMINARY MODEL TESTING
OF SLOPED BACKFILLS AND
DENSE FILL RETAINING WALLS

for

FALCONBRIDGE LTD.

by

R. J. Mitchell P.Eng.
Kingston GeoResearch Ltd.

October 1987

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2. Analysis of sloped fill 2

3. Dense fill model results 3

4. Analysis of dense fill retaining walls 4

5. Correlation of model results and analysis 5

6. Summary and recommendations 7

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Appendix B - Data from models
X1 to X6
- UCS data

Preliminary model testing of sloped backfills
and dense fill retaining walls
(an addition to backfill alternatives contract)

Kingston, Ontario   28 October 1987

1. Introduction

On the request of Mr. Philip Hopkins, Mines Project Engineer, (letter of 25 June 1987) several preliminary centrifuge models of sloped backfills and dense fill retaining walls were carried out. These were designed according to sketches provided and reproduced as Figure 1, Appendix A. The results are reported herein.

Sloped fill model results

Two sloped fill models were tested, giving the following results:

Test No.	T:C ratio	H model cm	UCS kPa	RPM at failure	λ	Hp m	$\frac{UCS}{\gamma H}$	Hvert 2UCS/γ
X1	40:1	27	76	200	120	33	0.195	8m
X2	30:1	27	120	242	176	47	0.134	13m

The raw data from these tests are contained in Appendix B and photographs of the failures appear on Figure 2, Appendix A.

From the photographs and tabulated data it is noted that (1) overall failure did not occur at the stress levels modeled but, rather, a base or toe spalling developed and (2) the sloped backfill prototype height (Hp) averaged about 4 times the predicted critical height of a vertical face (H_{vert}).

2. Analysis of sloped fill

Figure 3 shows the two potential upper bound modes of failure - slip on a plane at 45° and circular arc sliding - these are analyzed below:

a) For sliding on 45° plane, the safety factor, F, is

$$F = \frac{(UCS/2)\sqrt{2}H}{(W45 - MSC)\sin 45°} = \frac{0.707(UCS)H}{0.352H^2\gamma(0.707)}$$

Then, for F = 1, $H_c = 2.84\frac{UCS}{\gamma}$ ————(S1)

Compared to Hc = 2UCS/γ for vertical fill face.

(b) For rotational sliding, the safety factor, F, is

$$F = \frac{1.3H(UCS/2)1.07H}{(W_{arc} - MSC)0.74H} = \frac{0.70}{0.27}\frac{(UCS)H^2}{\gamma H^3}$$

Then, for $F = 1$, $H_c = 2.6 \; UCS/\gamma$ ---- (52)

The above indicates that a circular arc failure may be more critical than a simple sliding failure but that sloping the fill face over the upper two thirds of the fill height should increase the stability by a factor of between 1.3 and 1.4.

3. Dense fill model results

The two initial models of dense fill were cast, as shown in Figure 1, at 8:1 and 12:1 T:C. These proved to be too strong for testing in our centrifuge and two additional models were cast and tested using 30:1 T:C (with and without fibre reinforcing). The results are tabulated below:

Test No.	T:C ratio	H model cm	UCS kPa	RPM at failure	N	Hp m	$\frac{UCS}{\gamma H}$
X3	8:1	27	1015	260++	203	55++	N/A
X4	12:1	27	595	250+	188	51+	0.61
X5	30:1(F)	27	227	132	52	14	0.85
X6	30:1	27	173	110	36	9	1.0

Raw data from these tests are included in Appendix B and photographs after testing are reproduced in Figures 4 and 5, Appendix A. It is noted that model X3 showed very minor cracking at the test limit while model X4 showed substantial cracking. Models X5 and X6 failed in apparent shear.

4. Analysis of dense fill retaining walls

As indicated on Figure 6, potential failures may vary between about 60° and 45°. The safety factor, F, is

$$F = \frac{L \frac{UCS}{2} + W_1 \cos\alpha \tan\phi}{(W_1 + W_2) \sin\alpha}$$

For ease of solution, let $W = W_1 + W_2$ and assume $W_1 = W_2$

Then $F = \frac{L\,(UCS)/2}{W \sin\alpha} + \tan\phi / \tan\alpha$

The value of $\tan\phi / \tan\alpha$ can be assumed to be 0.3.

For $F = 1$, $F - (\tan\phi / \tan\alpha) = 0.7$ and

$$0.7 = \frac{d \tan 30}{W \sin\alpha \cos\alpha} \frac{UCS}{2}$$

Since $W = \gamma\,[d(1 + \tan 30 \tan\alpha)]^2 (\cot\alpha)/2$

$$0.7 = \frac{UCS}{\gamma} \frac{2 \tan 30}{d\,(1 + \tan 30 \tan \alpha)^2 \cot \alpha \sin 2\alpha}$$

The safety factor is a minimum for this approximate solution when $2\alpha = 90°$, $\alpha = 45°$, giving the strength requirement

$$UCS = 30d \qquad \text{- - - - - (58)}$$

This solution indicates that the strength should increase with depth (as with the free standing wall design) and the UCS given is the average over the failure plane. For a constant strength wall, the critical plane is that which passes at 45° through the base point (see Figure 6) and the critical height is

$H_c = \frac{UCS}{30}$, giving H_c = 7.5m and 5.6m for models X5 and X6 respectively.

It is clear that the centrifuge models are more stable than the analytical results would predict and it is interesting to explore the reason(s) for this. Some stability comparisons are tabulated below:

Model type	Model No.	T:C ratio	Prototype Ht. from model test	Prototype Ht. from analyses	Model Ht./ analytical Ht.	
sloped	X1	40	33m	10.4m	3.2	avg. = 3
sloped	X2	30	47m	16.4m	2.9	
dense wall	X3	8	55m**	33.6m	1.6**	
dense wall	X4	12	51m*	19.6m	2.6*	avg. = 2
dense wall	X5	30	14m	7.5m	1.9	
dense wall	X6	30	9m	5.6m	1.6	

Graphical comparisons are shown on Figures 7 and 8. Figure 7 shows that the model tests give a relationship of critical height (H_c) to strength (UCS) of $H_c = 7 \frac{UCS}{\gamma}$ for the sloped backfill tested. This is about three times better than expected from analyses and there is no readily apparent explanation for the major improvement in stability as a result of sloping the upper two-thirds of the backfill exposure. It is apparent that further testing of this new backfill alternative should be undertaken. Figure 8 indicates that the dense wall fill retaining structure can be adequately designed using the free standing wall approach (i.e. as if there was no retained fill) for heights up to about 10m. For greater heights, the free standing wall equation ($UCS = H\gamma$) is considered conservative and even greater economics could be realized. A 600 kPa (12:1 T:C) dense fill retaining wall at 60° appears to be capable of supporting 50 m of sand backfill. The reason for this better than predicted performance may be due to the reduction of lateral fill pressure due to arching. Figure 9 shows the measured fill pressures at transducers 4 and 5 which were mounted in the support wall in the strongbox. Although the measured pressures in models X5 and X6 are between the two theoretical predictions, other measurements show very low values of lateral pressure. Models X1 and X2 had cemented fill in contact with the transducers while models X3 to X6 had a compacted uncemented fill of a slightly lesser density. The reduced fill pressures in models X1 and X2 could be due to basal sliding (lack of lateral restraint) but this should not affect the failure in these models. The reduced fill pressure in models X3 and X4 could be due to lateral arching between the sloped dense fill face and the transducer support plate (rather than to the expected transverse arching). This lateral arching would not occur in the prototype since the uncemented fill could be more extensive. Clearly additional model testing using lower model heights and/or greater model lengths should be conducted.

6. Summary and recommendations

The two innovative backfill alternatives modeled - sloped fills and dense fill walls - are both very exiting backfill alternatives at this time, giving stable heights from preliminary model tests of 3 and 2 times, respectively, those from analytical predictions. It is quite clear that both of these alternatives could produce backfill cost reductions. Preliminary design criterion have been obtained from this early work as follows:

For sloped faces, $H_c = 7\ UCS/\gamma$

For dense walls, $H_c = UCS/\gamma$

These criterion should only be used, of course, for cost comparisons at this time and it is recommended that further centrifuge model studies be carried out in order to confirm and improve these preliminary findings.

We would be pleased to discuss any details of this work with you.

Yours very truly,

Robert J. Mitchell, P.Eng.

Kingston GeoResearch Ltd.

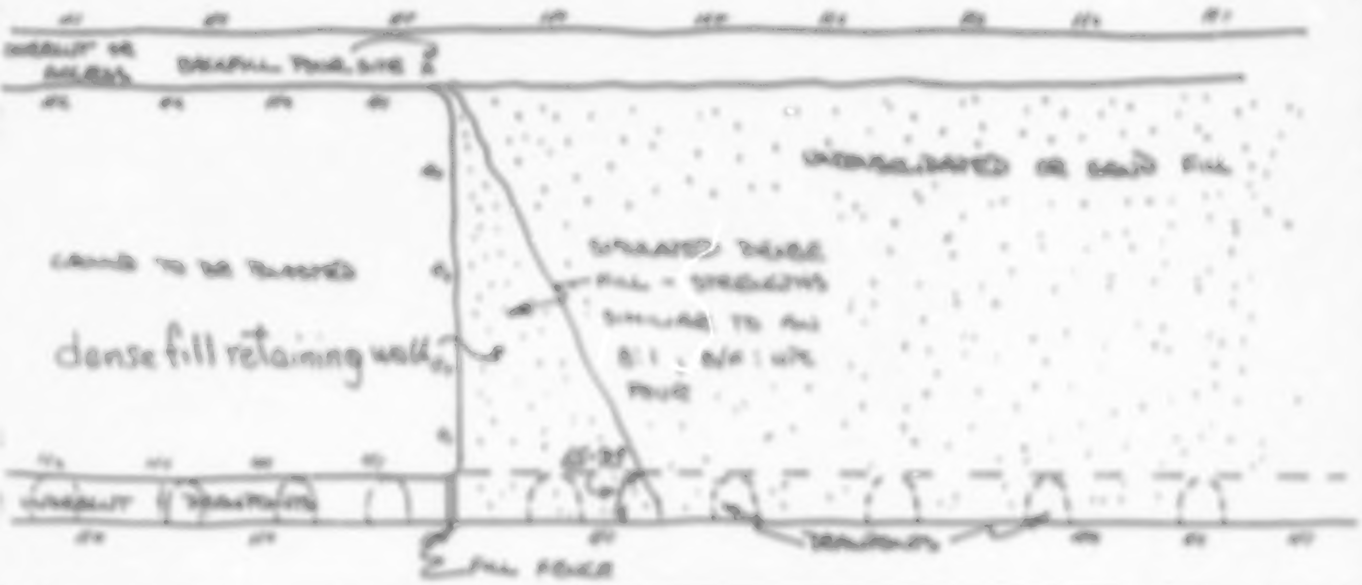
Appendix A - Figures 1 to 9

Failure plane & cracking, model X6

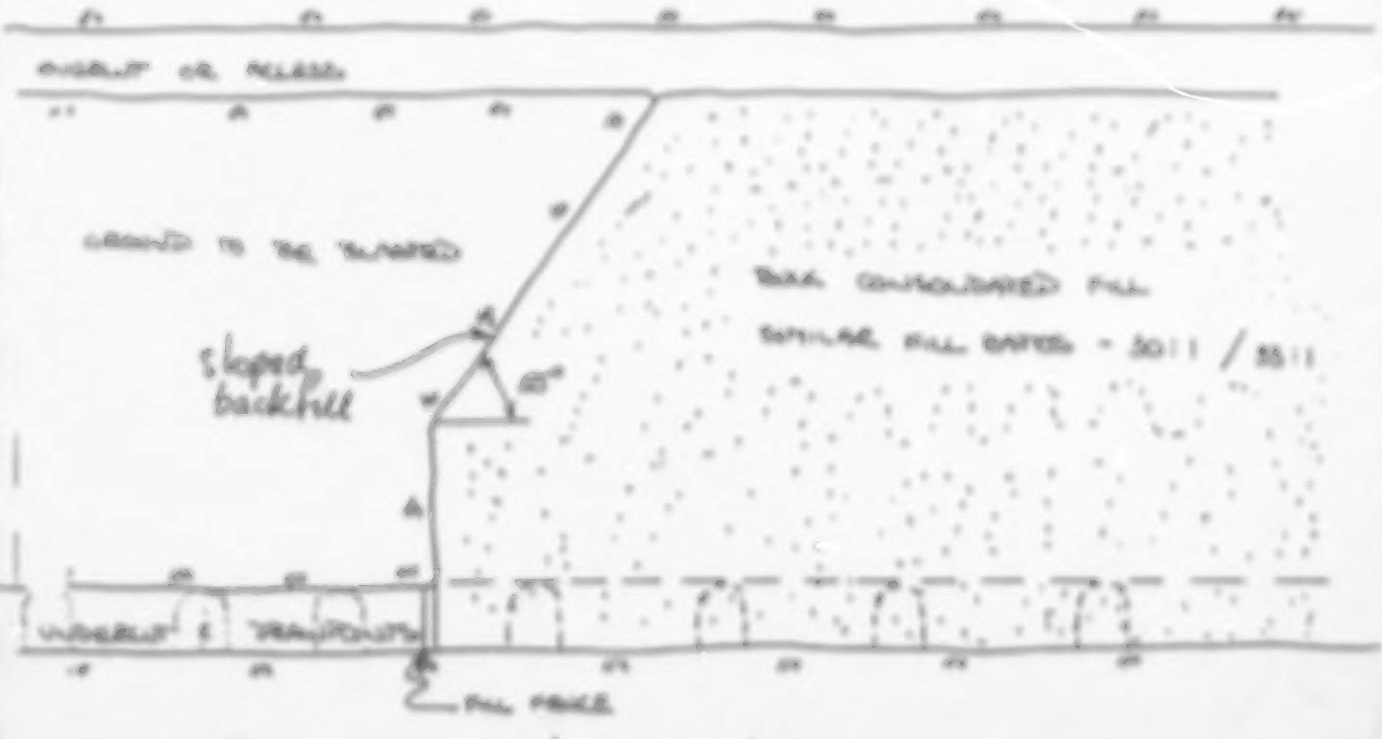
JUNE 4, [illegible]

- SKETCHES OUTLINING TWO ADDITIONAL CENTRIFUGE TESTS TO BE RUN BY KINGSTON GEORESEARCH LTD UNDER THE "PRESENT SUB-CONTRACT" INVOLVED IN DSS # [illegible]

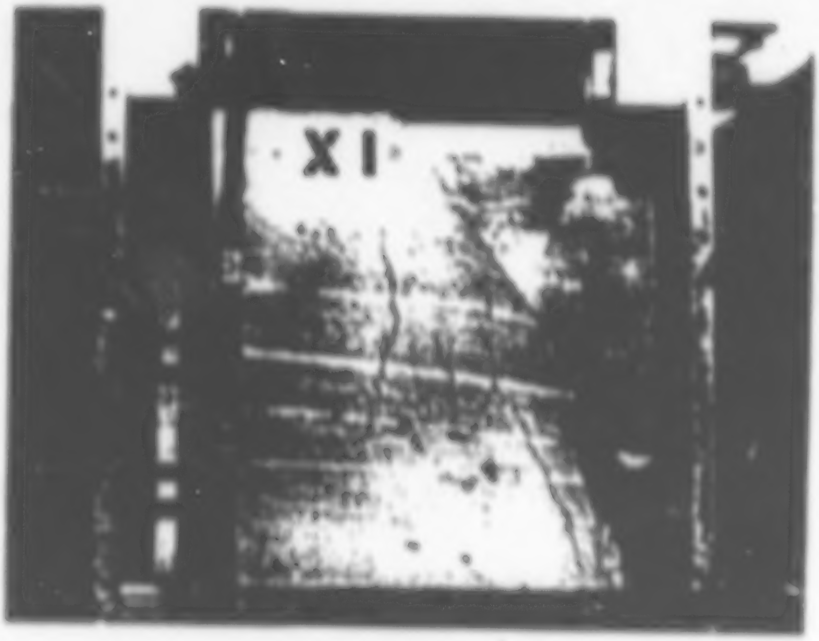
ADDITIONAL TRIAL # 1 - TWO SAMPLES TO BE TESTED


UNDERCUT OR ACCESS
BACKFILL POUR SITE
UNCONSOLIDATED OR SAND FILL
GROUND TO BE BLASTED
CEMENTED DENSE FILL - STRENGTHS SIMILAR TO AN 8:1 [illegible] POUR
dense fill retaining wall
UNDERCUT & DRAWPOINTS
DRAWPOINTS
FILL FENCE

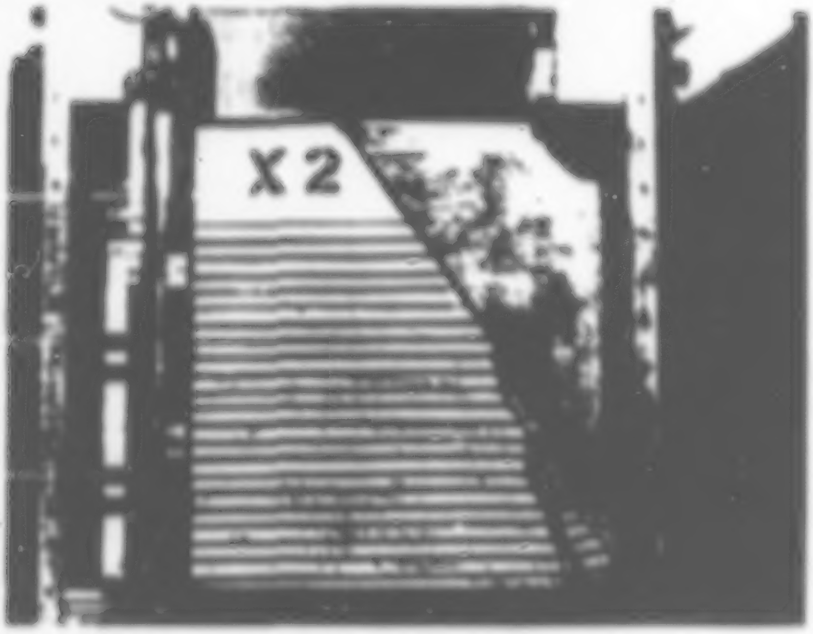
ADDITIONAL TRIAL # 2 - TWO SAMPLES TO BE TESTED


UNDERCUT OR ACCESS
GROUND TO BE BLASTED
[illegible] CONSOLIDATED FILL
SIMILAR FILL RATES - 30:1 / 35:1
sloped backfill
60°
UNDERCUT & DRAWPOINTS
FILL FENCE

[illegible]



(a) Failure in Model X1, 40:1 T:C



(b) Failure in Model X2, 30:1 T:C

FIGURE 2 PHOTOGRAPHS OF MODELS X1 & X2

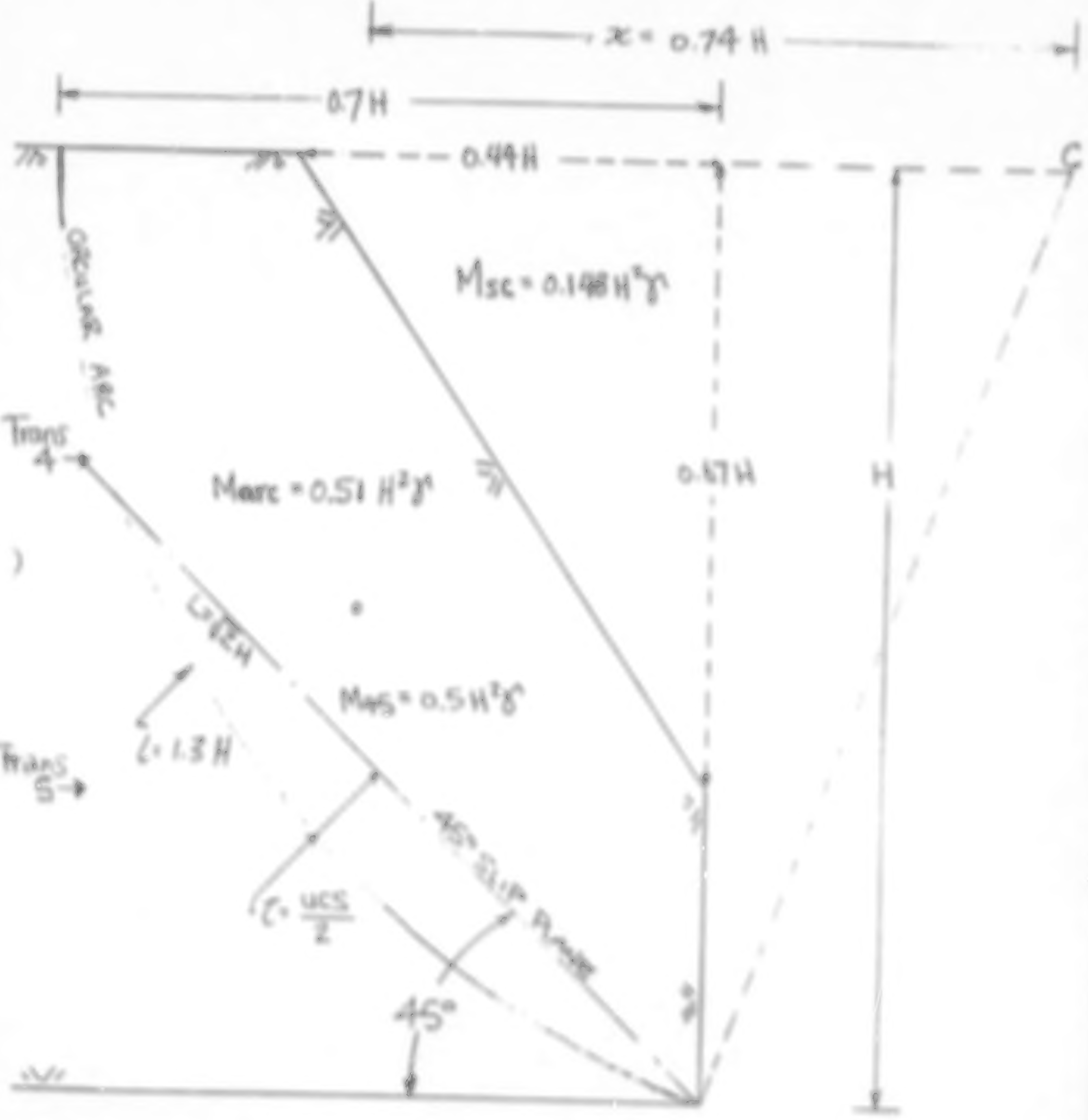
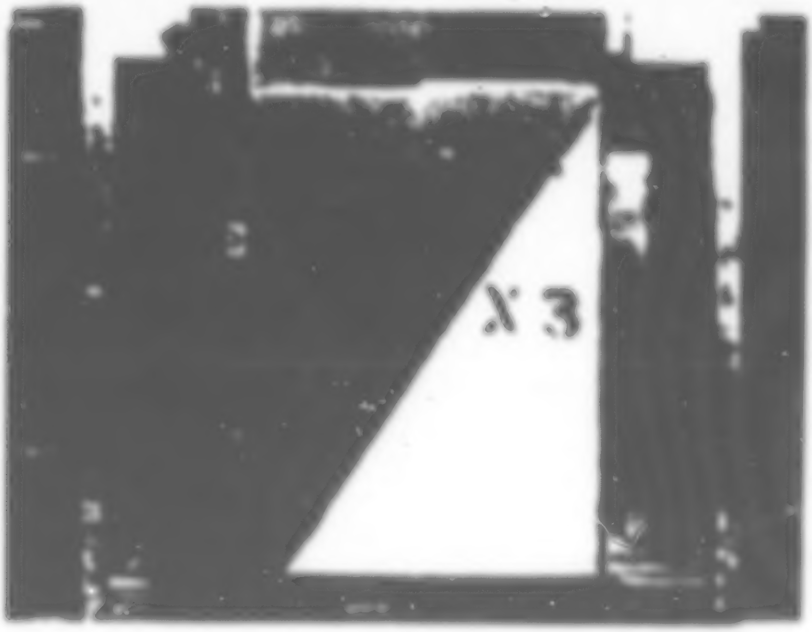
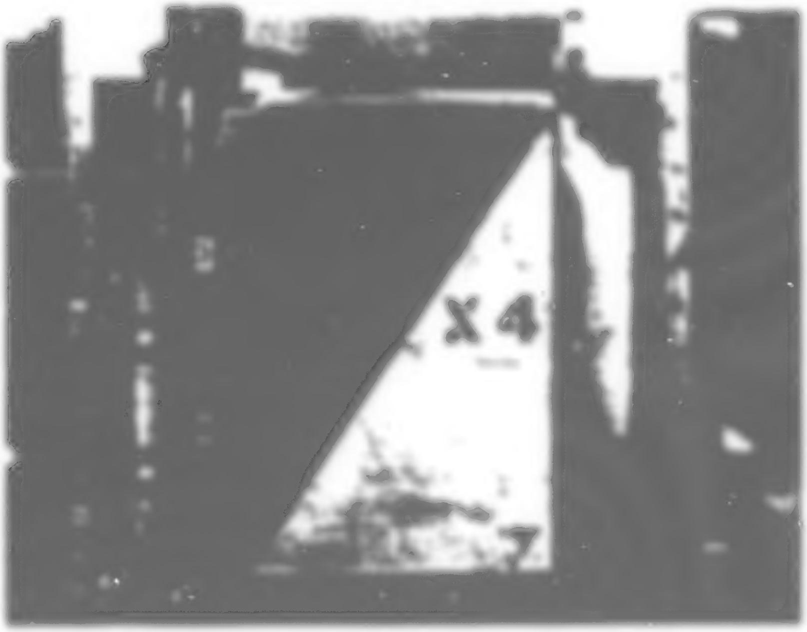

x = 0.74 H
0.7 H
0.44 H
C
CIRCULAR ARC
$M_{SC} = 0.148 H^2 \gamma$
Trans 4
$M_{arc} = 0.51 H^2 \gamma$
0.67 H
H
L=1.2H
$M_{45} = 0.5 H^2 \gamma$
L = 1.3 H
Trans 5
$c = \frac{UCS}{2}$
45° SLIP PLANE
45°

FIGURE 3 ANALYSIS OF SLOPED FILL FACE

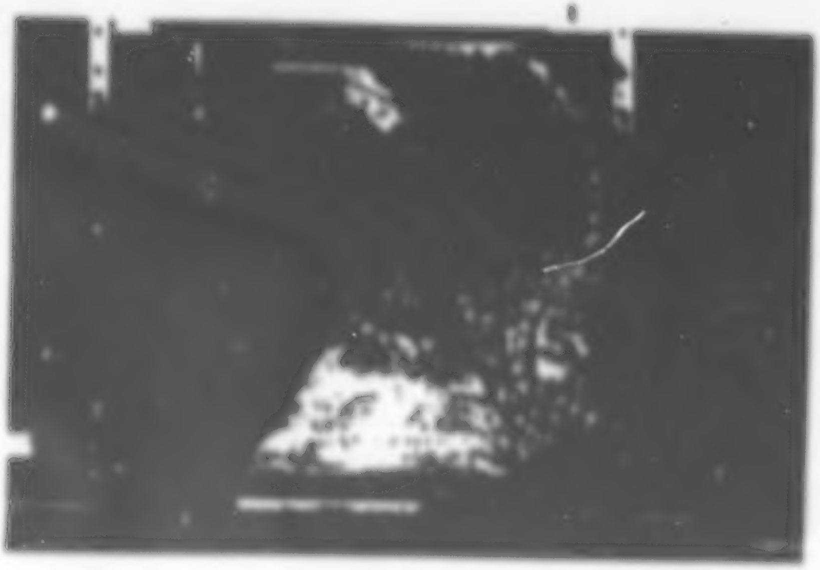


(a) Stable model X3, 8:1 T:C

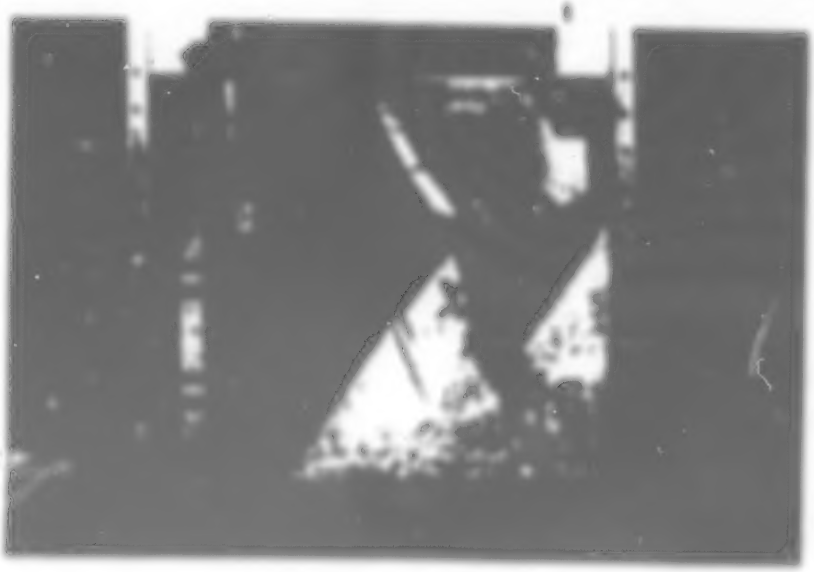


(b) Cracking in model X4, 12:1 T:C

FIGURE 4 PHOTOGRAPHS OF MODELS X4 & X5



(a) Failure in model X5, 30:1 T:C (F)



(b) Failure in model X6 30:1 T:C

FIGURE 5 PHOTOGRAPHS OF MODELS X5 & X6

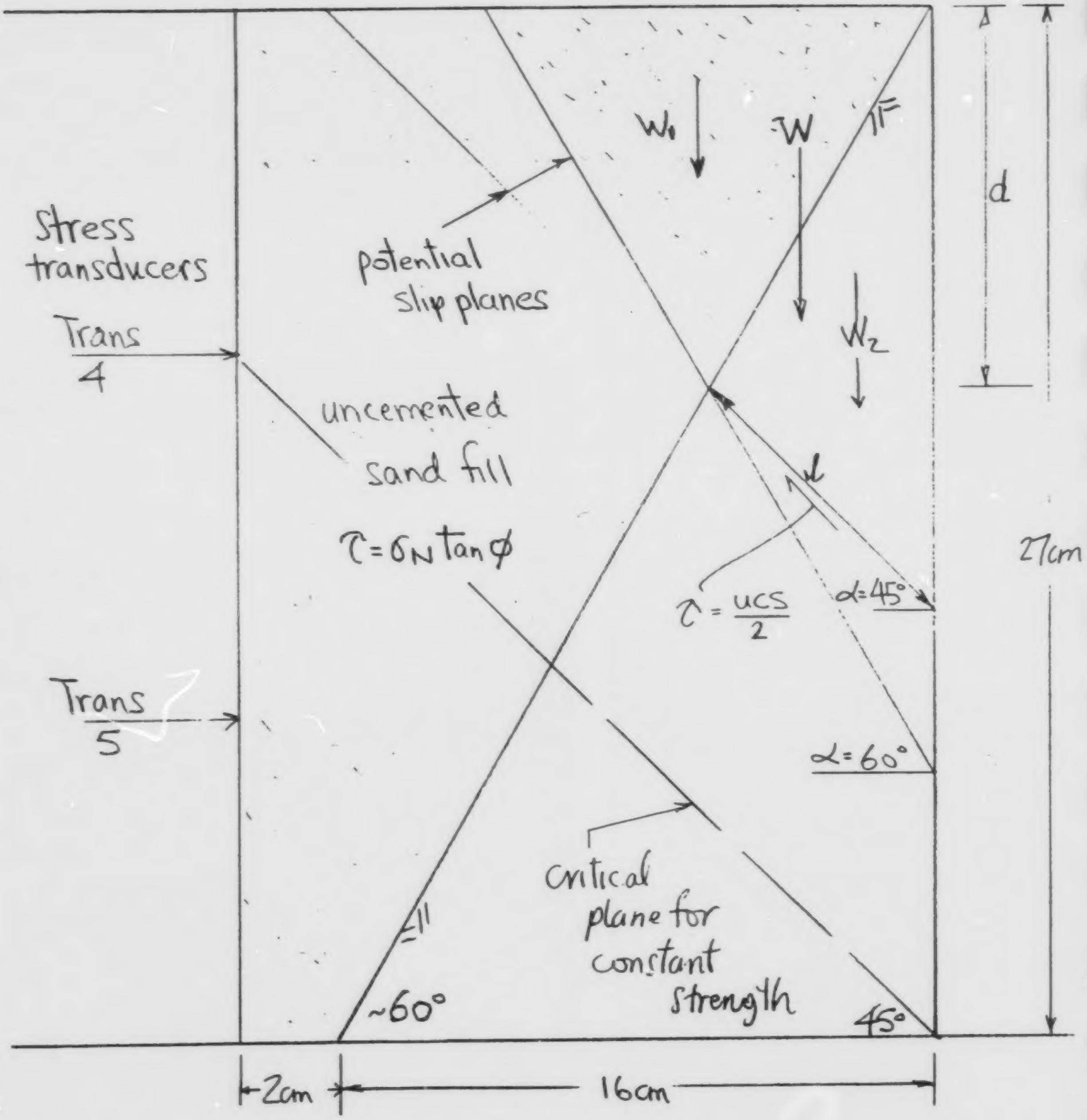

Stress transducers
Trans 4
Trans 5
potential slip planes
uncemented sand fill
$\tau = \sigma_N \tan\phi$
critical plane for constant strength
W_1
W
W_2
F
d
27cm
$\tau = \frac{UCS}{2}$
$\alpha = 45°$
$\alpha = 60°$
~60°
45°
2cm
16cm

FIGURE 6 ANALYSIS OF DENSE FILL WALL

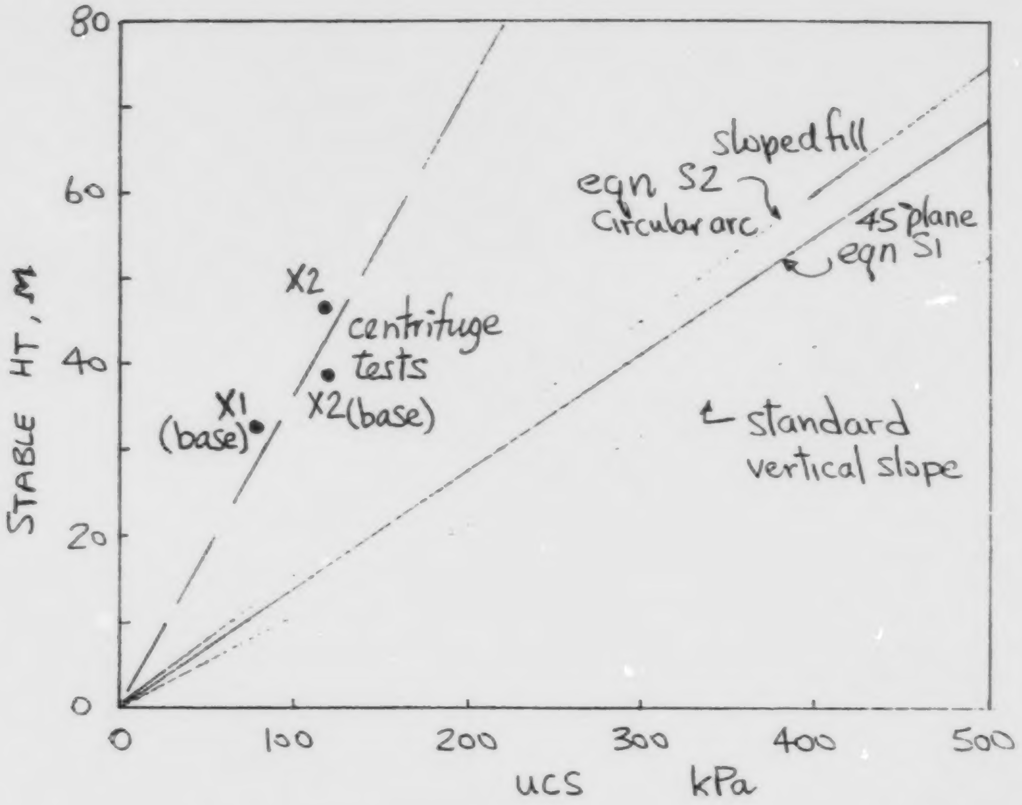


FIGURE 7 STABILITY OF SLOPED FILL FACES

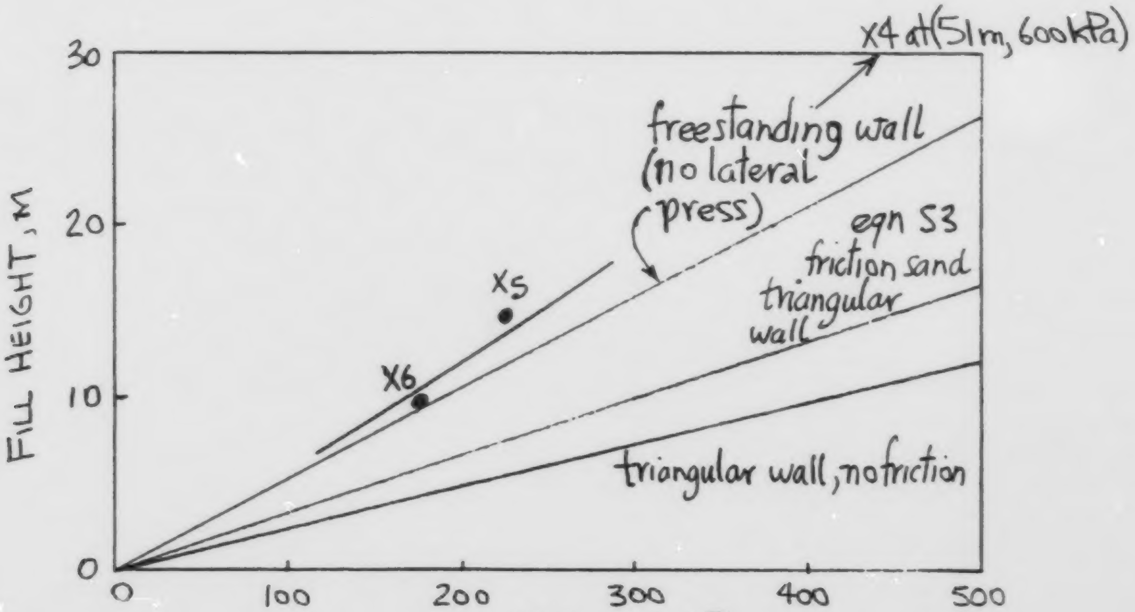


FIGURE 8 CEMENTED WALL STABILITY

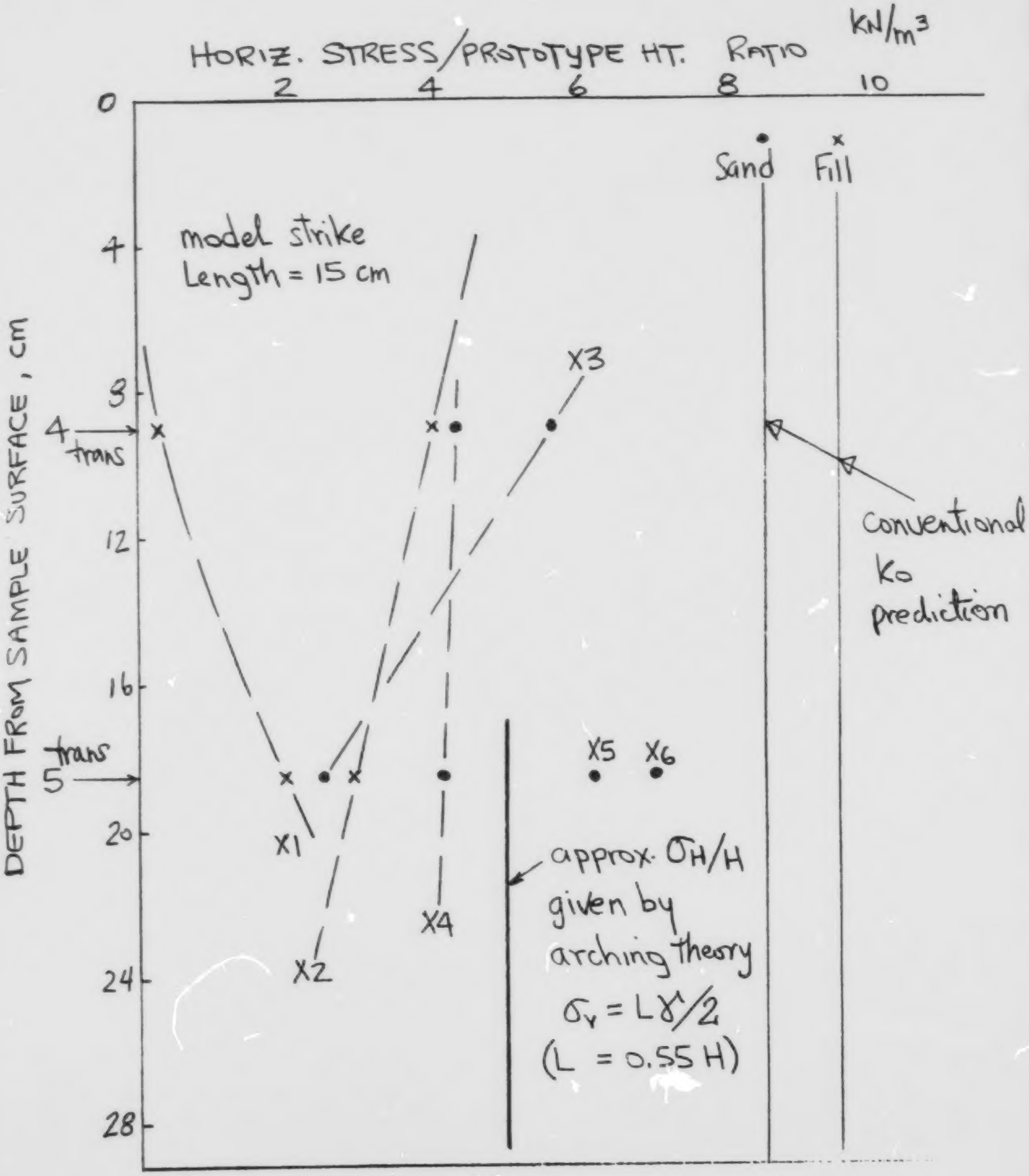


FIGURE 9 STRESS DISTRIBUTIONS IN BACKFILLS

APPENDIX H

LABORATORY AND FIELD PROCEDURES FOR DETERMINING THE PROPERTIES OF BACKFILL

1. Laboratory strength analysis - unconfined and triaxial compression.
2. Laboratory direct shear strength analysis
3. Laboratory percolation rate analysis
4. Laboratory density analysis
5. Laboratory sieve size analysis
6. Laboratory compressibility analysis
7. Laboratory determination of moisture content and liquid limit conditions
8. Physical modelling of failure mechanisms
9. In situ physical sampling techniques
10. In situ modulus analysis using pressuremeters/ dilatometers
11. Concrete slump test

Procedures

for

"Undrained" Hydraulic Fill Uniaxial - Triaxial Testing

1. MATERIALS

An estimate of the number of samples and test dates was determined from this estimate for the volumes of fill, cement and additives.

Fill: Enough fill is collected dried, (to a point where the material appears 'fluffy' when handled) coned and quartered and bagged in suitable sized samples for ease of handling. For consistent results this is one of the most important steps.

Binder: Again enough material should be collected so the entire series of the testing can be done using the same material. This too should be separated into representative size samples.

Additives: Same as cement, ex: slag, flyash, lime, sand, etc.

2. 3" X 6" TEST CYLINDERS

3" ABS plastic or like material is cut to length to make sure proper height-length ratio is maintained. See Fig. I (A).

When plastic pipe is cut to length, it is split lengthwise so that the pipe can be opened up to make the removal of the hardened sample easier. See Fig. I (B).

The bottom of the plastic cylinder is made up of a piece of glass (single diamond) cut to the same OD as the pipe. See Fig. I (C).

The plastic cylinder is now ready to be filled when the glass bottom is in place, held on by duct tape (2"-3" width - warehouse stock). The split in the pipe is covered with a strip of duct tape.

A collar is put on the top of the cylinder with a length of 3" duct tape. See Fig. I (C). This is essentially a collar extension. It extends the cylinder length by about 2 inches if the cylinder is now 8 inches long (6 inch cylinder +2" extension).

The interior of the plastic cylinders are coated with a thin coating of mineral or machine oil. This acts as a release agent so that the sample can be more easily removed from the test cylinder.

3. MIXING

- sample ratios determined beforehand
- material weighed out (to accuracy of scale)
- T.F. measured for % moisture, volume of water to be added is determined from this reading (use speedy moisture sampler)

When mixing, add water first

- then cement
- then additives
- then tailings fill
- mix 5 min. (slow, steady agitation)

4. CASTING SAMPLES

When material mixed - keep slurry material in suspension and ladle out to fill only a portion of each cylinder at a time, ie. fill several sample cylinders at one time approximately 1/3 at a time one after the other, then 2/3, then 3/3, then 4/3 (collar) before setting aside to set. Stir each sample with a rod to make certain there are no layers and material is homogeneous from top to bottom.

Let set 2-3 hours, decant water, remove collar and strike the tops levels with top of cylinder with a knife or other straight edge. Store overnight at room temperature (40° - 70°). Move samples to proper long-term storage location after 24 hours. Cover with plastic bag(s) and make certain they are kept moist.

5. TESTING

The day before testing, remove samples from cylinder, level and square ends, use dial gauge to make sure ends are parallel (to within .001 inches). Record all pertinent data, ie. sample no. date, etc. Let stand 6-8 hours so that the surface dries, cover before leaving as before.

Day of Test: Remove samples, record any further data,

- test - loading rate about 7 lb./sec.
- if material is to be saved for chemical testing, scrape samples from area of failure, place on sample bag and tag.

M. Beaudry
MB:lcm
Sept. 19/89

CYLINDER PREPARATION

Fig. A

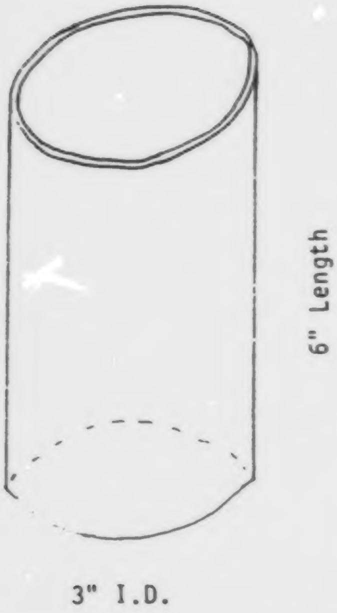

6" Length
3" I.D.

Fig. B

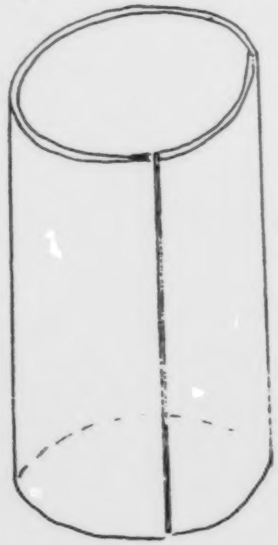
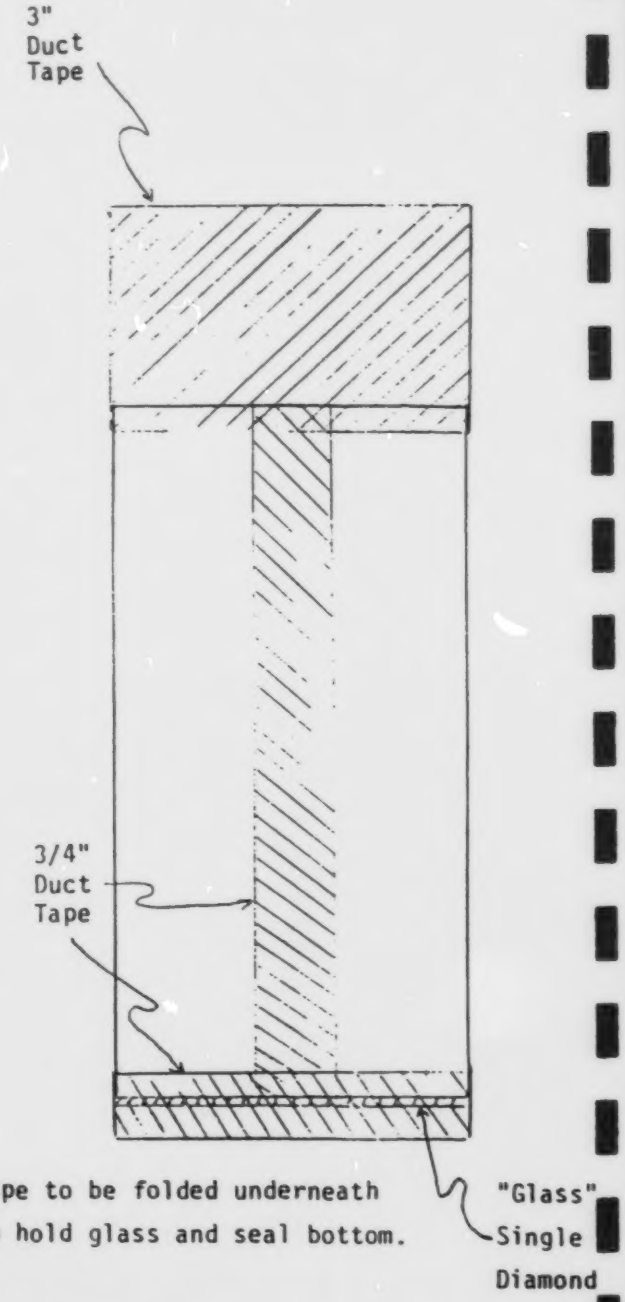


Fig. C


3"
Duct
Tape
3/4"
Duct
Tape
"Glass"
Single
Diamond

Tape to be folded underneath to hold glass and seal bottom.

PROCEDURES

FOR

"DRAINED" HYDRAULIC FILL UNIAXIAL - TRIAXIAL TESTING

1) Materials

An estimate of the number of samples and test dates was determined from this estimates for the volumes of fill, binder and additives was determined.

Fill: Enough fill is collected dried, (to a point where the material appears "fluffy" when handled) coned and quartered and bagged in suitable sized samples for ease of handling. For consistent results this is one of the most important steps.

Binder: Again enough material should be collected so the entire series of the testing can be done using the same material. This too should be separated into representative size samples.

Additives: Some as cement up: slag, fly ash, lime, sand, etc.

2) 3" x 6" Test Cylinders

3" ABS plastic or like material is cut to length to make sure proper height-length ratio is maintained. (See Figure 1-A)

When plastic pipe is cut to length, it is split lengthwise so that the pipe can be opened up to make removal of the hardened sample easier. (See figure 1-A)

The bottom of the plastic cylinder is made up of a piece of screen, cut 1" larger than the OD of the plastic pipe. (See figure 1-B)

The bottom screen is held on by strip of duck tape. The split in the pipe is also covered with a strip of duct tape.

A collar is put on the top of the cylinder with a length of 3" duct tape. (See figure 1-B). this is essentially a collar extension. It extends the cylinder length by about 2 inches if the cylinder is now 8 inches long (6 inch cylinder + 2" extension)

3) Proper drainage (to stimulate stope drainage).
A box with six inches of "damp" fill or (same material as what is being tested) is needed to placed cylinders before casting. (See figure 1-C). The bottom of cylinders are to be placed 3" deep in sand or fill.

After 24 hours of drainage at room temperature, remove collar and strike the top level with top of cylinder using a knife or other straight edge. Then after remove screen at bottom of cylinder, repeat same procedure as used for the top to bring it level with the bottom of plastic cylinder.

4) Mixing

- Sample ratios determined beforehand.
- Material weighed out (to accuracy of scale)
- T.F. measured for % moisture, volume of water to be added is determined from this reading (use speedy moisture sampler)

When mixing add water first
- then cement
- then additives
- then tailing fill
- Mix 5 min. (slow, steady agitation)

5) Casting Samples

When material mixed - keep slurry material in suspension and ladle out to fill only portion of each cylinder at a time, i.e. fill several sample cylinders at one time approximately 1/3 at a time one after the other, then 2/3, then 3/3, then 4/3 (collar) before setting aside to set. Stir each sample with a rod to make certain there are no layers and material is homogeneous from top to bottom.

Let set 2 - 3 hours, decant water, remove collar and strike the tops level with top of cylinder with a knife or other straight edge. Store overnight at room temperature (40° - 70°F). Move samples to proper long term storage location after 24 hours. Cover with plastic bag(s) and make certain they are kept moist.

6) Testing

The day before testing, remove samples from cylinders, level and square ends, use dial gauge to make sure ends are parallel (to with .001 inches). Record all pertinent data, i.e. sample No. date, etc. Let stand 6 - 8 hours so that the surface dries, cover before leaving as before.

Day of Test: Remove samples, record any further data,

- test - loading rate about 7 lb/sec
- if material is to be saved for chemical testing scrape samples from area of failure place on sample bag & tag.

SAMPLE PREPARATION

Fig. A

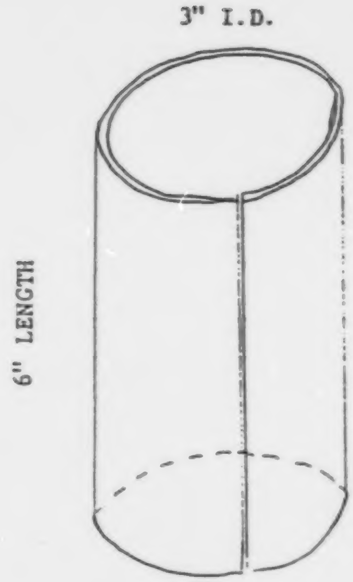

3" I.D.
6" LENGTH

Fig. B

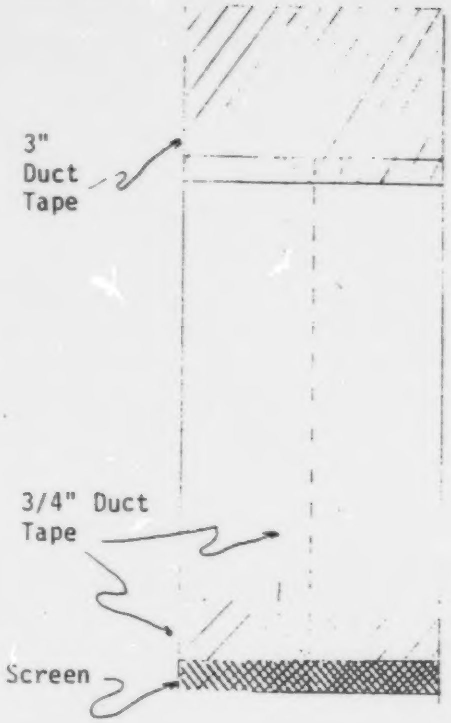
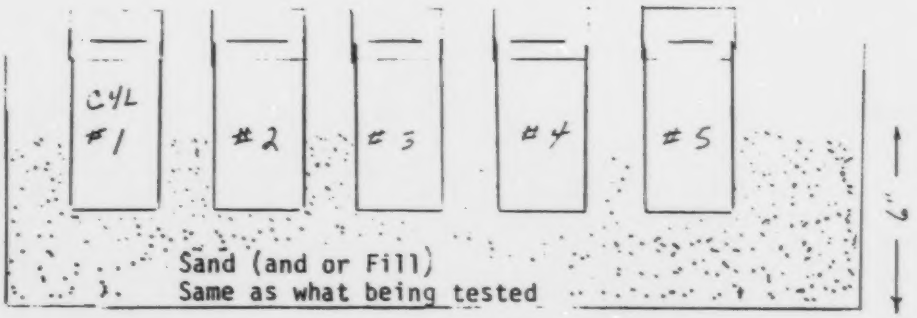

3"
Duct
Tape
3/4" Duct
Tape
Screen

Fig. C


CYL
#1
#2
#3
#4
#5
6"
Sand (and or Fill)
Same as what being tested

SAND BOX

TEST: Laboratory Strength Analysis - Unconfined and Triaxial Compression

EQUIPMENT NEEDS:

- PVC or ABS pipe, at minimum 5 cm diameter by 15 cm lengths
- Cement mixer or slurry tank
- Humidity controlled storage cabinet
- Compression loading frame (peak force capability at 100 kN) with triaxial confinement chamber at dimensions approximating twice and four times the sample height and diameter, respectively, to be tested
- Confinement chamber pressure gauge, peak capability = 450 kPa with resolution to ± 5 kPa
- Axial displacement gauges or transducers
- Extrusion frame for extracting cured samples from ABS or PVC casting tubes
- approximate equipment costs - $5000 - $10000 Cdn

GENERAL:

Two types of laboratory strength determinations are commonly applied, these being either unconfined compression or triaxial compression failure analysis. Sample preparation for both types of failure test is identical and consists of manufacture of cement consolidated aggregate materials within cylindrical molds which are permitted to moist cure for varying time intervals. Unconsolidated materials are very difficult to both manufacture and test in this manner, and are therefore tested using the direct shear method only.

Sample Preparation

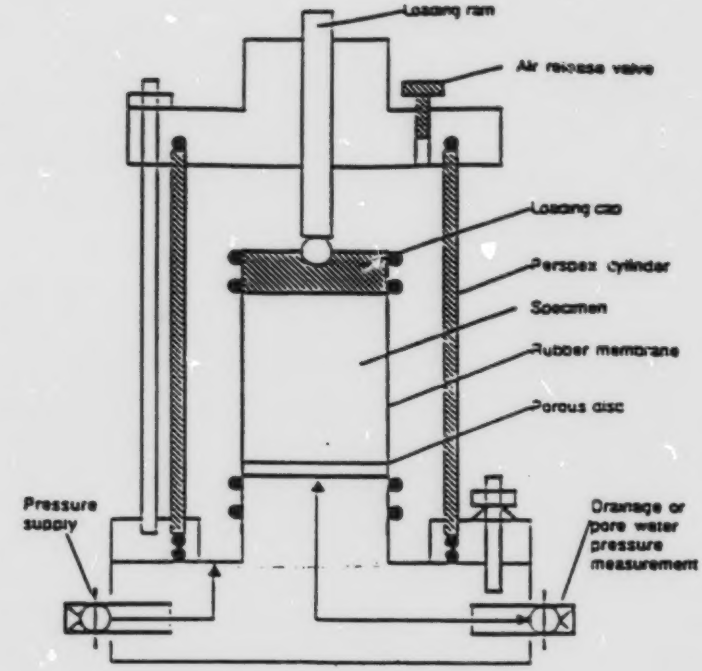
- a minimum of five samples of each mixture combination (tailings plus cement agent content), and more realistically, ten, must be prepared to yield sufficient failure data
- ASTM guidelines require that concrete testing cylinders be sized at 15 cm diameter by 30 cm long to insure that the sample cross-sectional area is at least 100 times the maximum aggregate cross-sctional area, thereby precluding frictional resistance and arching effects; on a similar basis, for typical tailings material sizes, tubes may be sized at 5 cm diameter by 15 cm long
- the 15 cm length permits casting of low density pulp materials while ensuring that final cured sample size will maintain the minimum 2-to-1 length-to-diameter ratio that is necessary for cylindrical test specimens; for coarser aggregate materials, larger diameter tubes should be selected
- casting tubes must be lightly greased to facilitate sample release and are placed on a suitable tray over which a burlap or paper towel layer has been placed to act as a porous draining surface during sample cure
- water is placed in the mixing chamber first; dry solids ingredients (tailings aggregates and cementing agents) are then added and allowed to mix for periods of between 10 and 20 minutes
- cylinders are filled by pouring the slurry pulp to levels suitable to prepare eventual 2-to-1 length-to-diameter specimens after completion of drainage
- After filling, mold samples can be densified by gently tapping the sides of the casting cylinders; this processs also permits release of any entrapped air within the slurry liquid
- after 24 hours, the cylinders and support tray on which they sit are placed within plastic enclosures or a humidity-controlled chamber and are permitted to undergo moist curing in a manner similar to that which occurs in-situ
- testing of samples should take place at standard intervals of 7, 14, 28, 56 and 112 days following initial casting; where delayed curing binding agent materials are being used (slag or fly ash) additional curing to 180, 250 and 360 day intervals may be necessary to test ultimate (long term) sample strengths

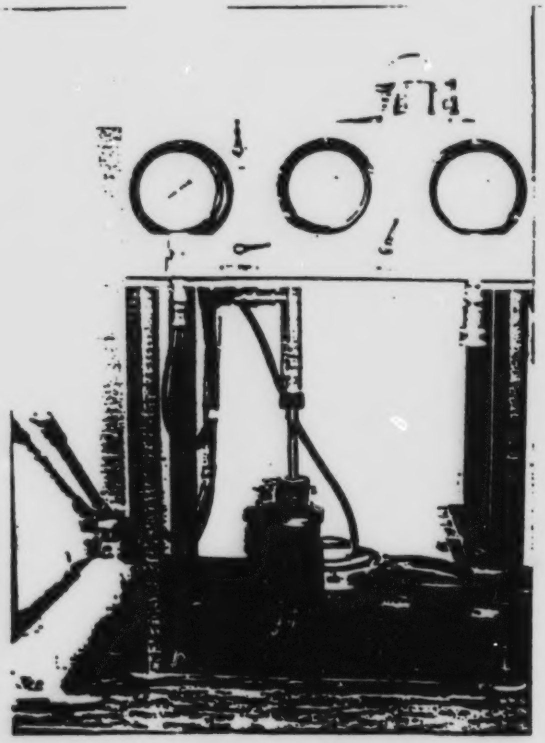
TEST METHOD: TRIAXIAL COMPRESSION STRENGTH ANALYSIS

1) The standard triaxial test assembly, illustrated in the following figure, is utilized to apply confinement to cylindrical test specimens while increasing axial stresses are applied to induce failure.
2) At a time one day prior to testing, samples are extruded from the confinement tubes, the ends are carefully trimmed (to prepare final 2-to-1 length-to-diamter aspect ratios) and are replaced within the humidity-controlled curing chamber
3) Samples are weighed and their dimensions are measured prior to testing to determine placed apparent bulk density values
4) Samples are then manually installed within latex rubber sheaths, with aluminum loading platens (of a similar diameter) being placed and held in contact with the sample ends
5) Specimens are placed and sealed within the triaxial confinement apparatus; with a small component of axial stress applied to the ends of the sample, confining stress is applied by pressurizing the confining fluid; for most backfill materials, limiting confinement stress in the order of 175 - 200 kPa will be suitable (for this reason, domestic water supplies may be considered for use as the triaxial confinement medium)
6) Where stronger backfill materials may be created, the level of confining stress may be increased through application of pressurized air to boost the limited domestic water supply pressure
7) Once the confining stress has been established, strain controlled axial loading is performed (at rates approximating 1 - 2% of sample length per minute) until specimen failure is indicated to occur
8) During load application, sample dilation (volumetric expansion) may occur, causing the confinement stress level to rise; by careful drain valve manipulation, the confining stress rise can be balanced and the stress level held constant.
9) At failure, both sample failure stress and confinement stress levels are recorded; where determination of sample elastic deformation modulus is desired, continuous readings of axially-applied force (from proving ring or load cell output) and axial deformation (from dial gauge or displacement transducer output) can be recorded and converted to stress/strain parametric data.
10) Axial loads are released and drainage of the confining liquid is permitted; samples are unwrapped from the latex covers and, where visible, visible failure plane angles are measured for comparison with subsequent derived Mohr-Coulomb failure data.
11) When all samples of specific mixtures (various tailings/cement ratio combinations after uniform cure intervals) have been failed, (axial failure stress and lateral confining stress) data is plotted in standard fashion to determine the Mohr-Coulomb failure parametric data

TEST METHOD: UNCONFINED COMPRESSION STRENGTH ANALYSIS

1) In order to determine unconfined material strengths, testing of prepared and moist cured cylindrical specimens can be conducted within the same loading frame and in a similar manner to that detailed for triaxial confinement tests, with the exception that no confining pressure will be applied
2) Failure axial stress data (unconfined compressive strength) can be plotted on triaxially-derived failure stress curves for purposes of data comparison or verification


Loading ram
Air release valve
Loading cap
Perspex cylinder
Specimen
Rubber membrane
Porous disc
Pressure supply
Drainage or pore water pressure measurement



- **Failed Uniaxial Specimen**

(Deformation Dial Gage NOT Shown)

TEST: Laboratory Direct Shear Strength Analysis

EQUIPMENT NEEDS:

- Shear box loading frame with split shell sample confinement cylinder at approximately 7 - 15 cm diameter
- Two hydraulic loading jacks, each at 30 kN force capacity, for application of normal and shear stresses to sample during failure tests
- Calibrated hydraulic pressure gauges or force transducers for shear and normal stress measurement
- Shear displacement dial gauge or displacement transducer apparatus to permit unrestricted drainage)
- Approximate equipment cost (total) - $3000 - $5000 Cdn

TEST METHOD:

1) The apparatus is cleaned and a layer of silicon grease is applied between the shear box halves
2) With the base porous stone in place, known weights of material, at moisture content conditions simulating drained conditions in-situ, are placed loosely within the shear cell
3) Normal preconsolidation stress is applied to a top loading platen to initially consolidate the material to density conditions typical of the in-situ fill mass when acted upon by known superincumbent loads
4) The preconsolidation stress is removed and the volume of sample is measured
5) Two loading platens, with linear thrust bearings between, are installed on top of the emplaced sample which is then subjected normal consolidation stress at a level equal to the initial preconsolidation stress level.
6) Shear stress is then applied to the top half of the shear box at rates of shear strain approximating 1 - 2% of the sample diameter per second. Shear stress is applied until no increase occurs, at which point both normal and shear stress values are recorded.
7) The preceding two steps are repeated but at a level of normal consolidation stress less than the preconsolidation stress level. Upon completion of the second shear trial, the sample will be discarded and another prepared.
8) A suite of samples, usually greater than five, will be tested in a similar fashion and the summary shear and normal stress data for failure will be recorded.
9) Shear and normal stress data will be plotted and the linear regression estimate produced from the sum of test data will infer the shear strength parametric conditions for the tailings material at the selected density and moisture content condition based upon standard Mohr-Coulomb failure theory.
10) Similar characterization tests may be performed on samples of cemented tailings which can be prepared and cast in shear box cells, then permitted to cure prior to testing. Such tests can be used to replace triaxial confinement failure tests, where equipment shortages may exist.

A typical laboratory shear frame assembly and schematic of the normal/shear stress loading conditions are illustrated in the following views.

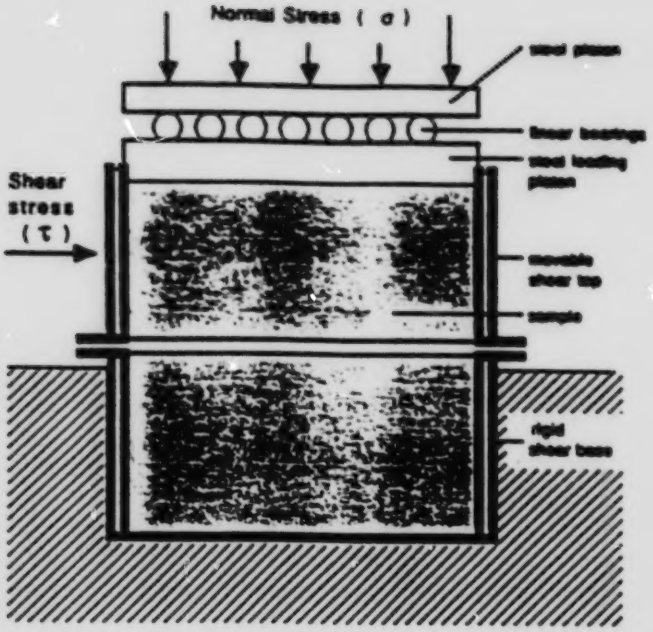
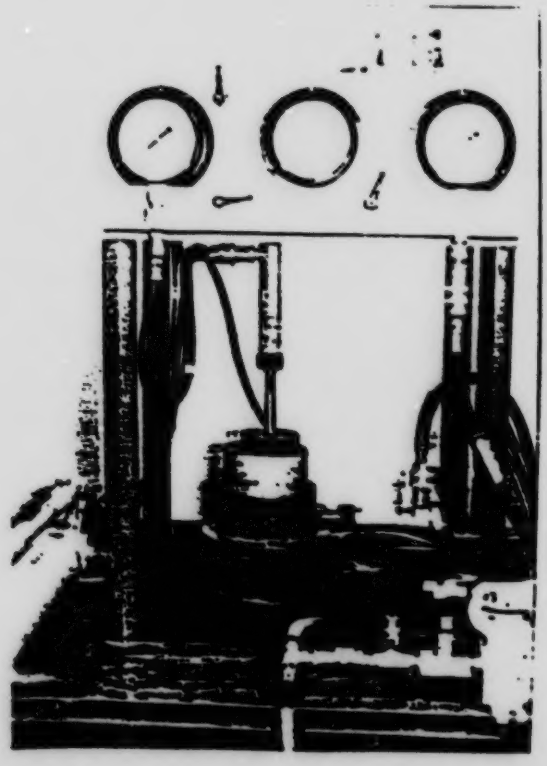

Normal Stress (σ)
steel piston
linear bearings
steel loading piston
Shear stress (τ)
movable shear top
sample
rigid shear base

FIGURE B-1 - Typical Laboratory Direct Shear Test Apparatus



Direct Shear Apparatus

FALCONBRIDGE LIMITED
SUDBURY OPERATIONS

LABORATORY PROCEDURES

FOR

METHOD OF DETERMINATIONS PERCOLATION RATE OF WATER THOUGH SOLIDS

METHOD OF DETERMINING PERCOLATION RATE OF WATER THROUGH SOLIDS

1. Percolation tubes used are 17 inches long by 3.35 cm. inside diameter. About 3 to 5 layers of burlap are fastened across one end of the tube by means of friction tape. A band of tape is wrapped around the tube 12¼ inches from the covered end.

2. The tube is filled with solids to the 12¼" inch mark. If dry solids are used it is preferable to add water and mix well in the tube, being careful not to overfill.

3. An Erlenmeyer flask with 1 hole stopper and glass tubing is inverted over the material in the tube. The end of the glass tubing must be cut at about 45 degrees so the water can run out of the flask and the tubing must be of sufficient length that 3 inches of water lie above the 12¼ inch marker.

4. Let water percolate through the material for at least 30 minutes before starting test. After test is started, collect water for 30 minutes and measure volume collected. With tailing fill, consecutive measurements at 30 minute intervals produce similar results. For other material this aspect of time interval should be investigated.

5. Percolation Rate (inches per hour) =

$$\frac{\text{Volume collected in 1 hour (cc} \times 0.06102 = \text{cubic inches)}}{\text{Cross-sectional Area of Tube in square inches}}$$

PERCOLATION TESTS

INTRODUCTION

The main purpose of the percolation test is to control the quality of the fill that is being produced in the mill for transportation to the stopes underground. The tests should be as simple and quick as possible, so that frequent testing is feasible and undesirable production can be quickly changed. For this purpose, the QUICK TEST, as described on pages 2 and 3, is proposed.

At the same time, for special studies and research on the fill material, it may be useful to have a testing procedure that will produce more accurate results. For example, the variation in permeability of the fill between the mill and the stopes might be significant. In this case, to obtain a valid measure of the effects of transportation (possibly decreasing the resultant permeability), it would be desirable to avoid introducing error arising from a factor such as the difference in temperature of the pulp water. For this more complex type of study, the LONG TEST, is more suitable.

QUICK TEST

Apparatus

1. Percolation tube approximately 50 cm long x 5 cm ID (18 in. long x 2 in. ID) with or without an overflow tube at 5 cm (2 in.) from one end. If no overflow tube is provided, wrap scotch tape around top to indicate 5 cm (2 in.) and 7.5 (3 in.) distances from the end.*
2. 500 ml Erlenmeyer flask with a perforated rubber stopper containing a length of glass tubing approximately 7.5 cm long x 1 cm OD (3 in. x 0.4 in.) with the end outside the flask cut at 45 deg. (This item is required if the percolation tube does not have an overflow tube.)
3. No. 100 mesh screening and rubber O-rings to fit over the bottom of the percolation tube.
4. 400 ml beaker.
5. 250 ml graduated cylinder.
6. Water supply tank, plastic tubing, funnel and spatula.
7. Distilled water (untreated water may be used if not too high in impurities).
8. Supporting frame and clamps.

Procedure

1. Attach the No. 100 mesh screening with the O-rings to the bottom of the percolation tube.
2. Fill the percolation tube with fill pulp either to within 1 cm (½ in.) of the overflow tube or to within 5 cm (2 in.) to 7.5 cm (3 in.) from the end of the tube if there is no overflow tube. Take the temperature.
3. If the percolation tube has an overflow, start water flowing into the tube from the water tank to maintain the head at the level of the overflow tube. If the percolation tube has no overflow, fill the Erlenmeyer flask with water, place the stopper and glass tube in the flask, and clamp in an inverted position over the percolation tube with the 45 deg. cut end 1 cm (½ in.) above the level of the fill. The water should be at a temperature as close to 20°C (68°F) as possible.
4. Allow about 30 minutes of percolation for settlement of the fill pulp in the percolation tube.
5. Empty the beaker and then collect the percolation water for a minimum of 30 minutes, timed as accurately as possible, measuring the water in the graduated cylinder,
6. Calculate the approximate percolation rate, or coefficient of permeability, k, from:

$$k = Q/A \text{ cm/min},$$

$$\text{or } K = 23.6\ Q/A \text{ in./hr},$$

where Q = flow rate in cc/min, A = cross-section area of fill sample in cm^2.

---

* The tube diameter could be as low as 32 mm (1-¼ in.) without significantly affecting results.

Precautions

1. Appearance of pulp should be checked for segregation; samples with any segregation should be discarded.
2. If the percolation tube is overfilled, discard the sample.
3. The No. 100 mesh screening should be washed after each test and discarded when its porosity is seriously decreased by unmovable particles.
4. If the temperature of the pulp is much different than 20°C (68°F), a correction can be made as in the LONG TEST. The reported value should be described as having been corrected.

LONG TEST

Apparatus

1. Percolation tube approximately 50 cm long x 5 cm ID (18 in. long x 2 in. ID), with or without an overflow tube at 5 cm (2 in.) from one end. If no overflow tube is provided, wrap scotch tape around top to indicate 5 cm (2 in.) and 7.5 cm (3 in.) distances from the end*.
2. 500 ml Erlenmeyer flask with a perforated rubber stopper containing a length of glas tubing approximately 7.5 cm long x 1 cm OD (3 in. x 0.4 in.) with the end outside the flask cut at 45 deg. (This item is required if the percolation tube does not have an overflow tube.)
3. No. 100 mesh screening and tubber O-rings to fit over the bottom of the percolation tube.
4. 400 ml beaker.
5. 250 ml graduated cylinder.
6. Water supply tank, plastic tubing, funnel and spatula.
7. Distilled water (untreated water may be used if not too high in impurities).
8. Supporting frame and clamps.
9. Balance (0.1 g sensitivity and a minimum of 1000 g capacity).
10. Drying oven (up to 200°C, with approximately 12 in. x 12 in. x 10 in. inside dimensions).
11. Hot plate.
12. Thermometer.
13. Stop clock.
14. Metre stick.
15. Rubber mallet.
16. 600 ml beaker.
17. 500 ml graduated cylinder.
18. Evaporating dishes of 765-, 1285-, and 2100 ml capacity.

* The tube diameter could be as low as 32 mm (1-¼ in.) without significantly affecting results.

Procedure

1. Attach the No. 100 mesh screening with the O-rings to the bottom of the percolation tube.
2. Fill the percolation tube with fill pulp at a temperature as close to 20°C (68°F) as possible, either to within 1 cm (½ in.) of the overflow tube or to within 5 cm (2 in.) to 7.5 cm (3 in.) from the end of the tube if there is no overflow tube.
3. If the percolation tube has an overflow, start water flowing into the tube from the water tank to maintain the head at the level of the overflow tube. If the percolation tube has no overflow, fill the Erlenmeyer flask with water, place the stopper and glass tube in the flask, and clamp in an inverted position over the percolation tube with the 45 deg. cut end 1 cm (½ in.) above the level of the fill. The water should be at a temperature as close to 20°C (68°F) at possible.
4. Allow about 30 minutes of percolation for settle of the fill pulp in the percolation tube. Mark the tube at the top of the sample with a grease pencil (the height of the material in the tube is required to calculate void ratio and density).
5. Select a graduated cylinder that will hold the flow of about 1 hour. Start the test by recording the time, water level in the graduated cylinder, and water temperature. Take readings approximately every 10 minutes.
6. After one test, lightly tap the percolation tube with the rubber-headed mallet. Again, after 30 minutes of percolation, mark the height of the fill on the tube and repeat Step 5.
7. Repeat Step 6, again increasing the density of the material, obtaining a third set of readings.
8. Turn off water and measure length of sample to each density level.
9. Allow remaining water to percolate from the tube for 24 hours.
10. Remove fill from the tube and weigh.
11. Dry fill at 110°C for 24 hours and weigh accurately. Calculate water content, w, by:

$$w = (T - W)/W$$

where T = total weight of water and solids, W = weight of solids.

12. Decant water from graduated cylinder; then remove (washing if necessary) the solids into a large evaporating dish and dry for 24 hours at 110°C. Weigh and express ratio of fines washed out, f, by:

$$f = F/W$$

where F = the dry weight of fines; W = the weight of solids in the percolation test.

13. Plot graduated cylinder readings (y-axis) versus time (x-axis); measure the average slope giving the flow rate in cc/min.
14. Calculate the percolation rate, or coefficient of permeability, k, from:

$$k = QL/(AH)/\text{cm/min.}$$

$$\text{or } k = 23.6\ QL/AH \text{ in./hr,}$$

where Q = flow rate in cc/min, L = length of sample in cm, A = cross-section area of sample in cm^2, H = head of water over bottom of sample in cm.

15. Correct percolation rate, or coefficient of permeability, for viscosity of water to give equivalent values at 20°C by multiplying by:

    1.54 if temperature is 4°C (40F)
    1.32 if temperature is 10°C (50F)
    1.12 if temperature is 16°C (60F)
    0.97 if temperature is 21°C (70F)
    0.86 if temperature is 27°C (80F)

16. Calculate dry density, in g/cc, at each level of density by dividing the dry sample weight by its volume. (To convert to pounds per cubic foot, multiply by 62.4).

17. Calculate the void ratio at each density:

$$e = \frac{GV}{W} - 1$$

where G = specific gravity of solids, V = volume of sample in cc, W = dry weight of sample in g.
(Porosity, n, can be calculated by: $n = e/(1 + e)$.)

18. Plot percolation rates, or coefficients of permeability, against void ratio and interpolate for the void ratio of the in situ fill underground.

19. Calculate the degree of saturation, S, after 24 hours in the percolation test by:

$$S = Gw/e,$$

where G = specific gravity of solids, w = water content (weight of water/ weight of solids), e = void ratio.

Precautions

1. Appearance of pulp should be checked for segregation; samples with any segregation should be discarded.

2. If the percolation tube is overfilled, discard the sample.

3. The No. 100 mesh screening should be washed after each test and discarded when its porosity is seriously decreased by unmovable particles.

4. If the water contains an excessive amount of air, the accuracy of the test might be diminished. The influence of air in the water can be determined by de-airing before the percolation test. This is done by heating the fill pulp sample slowly to a light boil. The sample is then allowed to cool at room temperature. It is then agitated and poured into the percolation tube for testing as above. A significant difference in results shows that the air content was excessive.

PERCOLATION RATE

TIME: 30 MINUTES

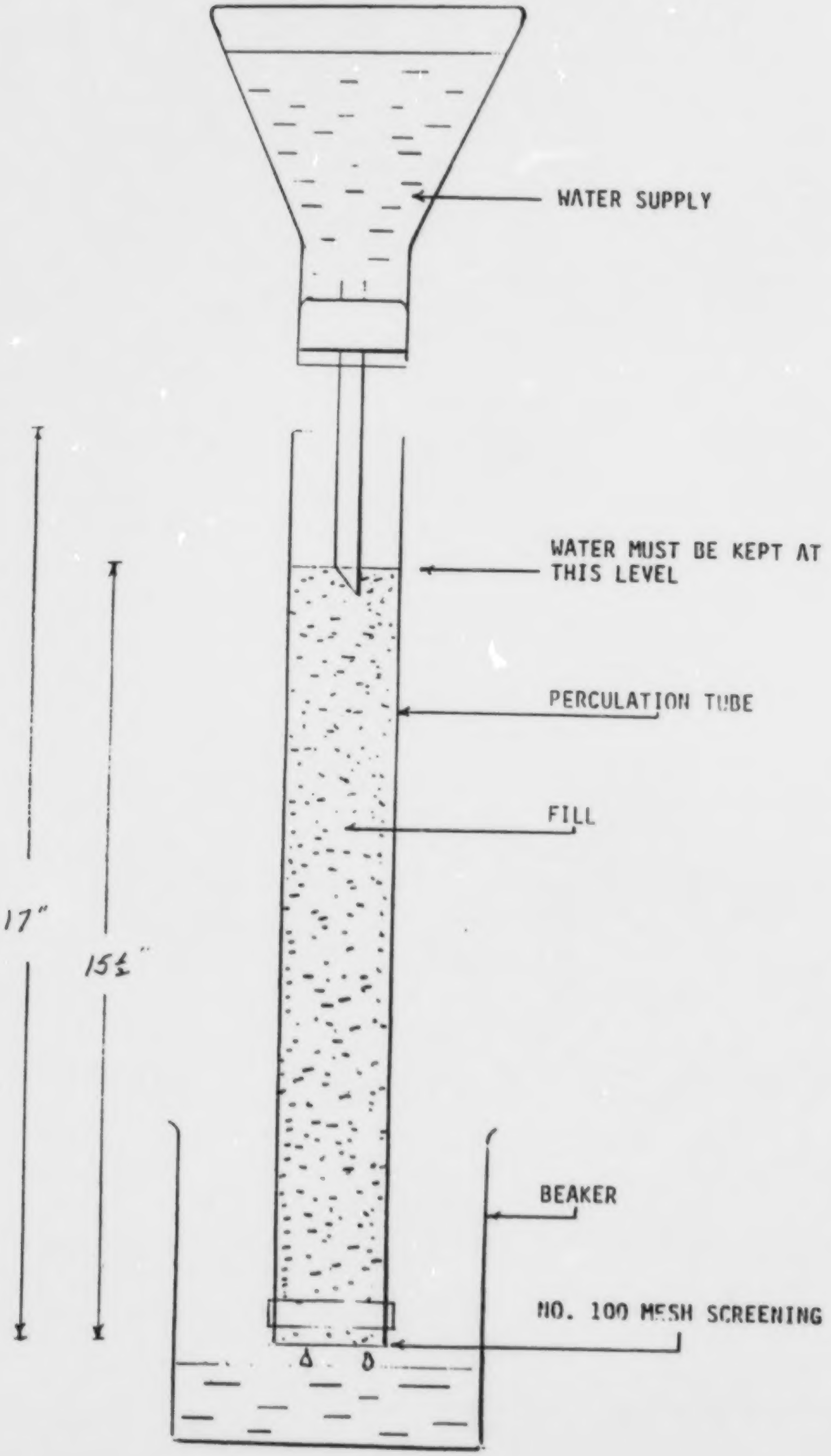
TUBE I.D. = 1.5 in. or
AREA = 11.4 cm^2

cc's	PERC. RATE	cc's	PERC. RATE
40	2.76	76	5.25
41	2.83	77	5.32
42	2.90	78	5.39
43	2.97	79	5.46
44	3.04	80	5.53
45	3.11	81	5.60
46	3.18	82	5.67
47	3.25	83	5.74
48	3.32	84	5.80
49	3.39	85	5.87
50	3.46	86	5.94
51	3.52	87	6.01
52	3.59	88	6.08
53	3.66	89	6.15
54	3.73	90	6.23
55	3.80	91	6.29
56	3.87	92	6.36
57	3.94	93	6.43
58	4.01	94	6.50
59	4.07	95	6.56
60	4.14	96	6.63
61	4.22	97	6.70
62	4.28	98	6.77
63	4.35	99	6.84
64	4.42	100	6.91
65	4.49	101	6.98
66	4.56	102	7.05
67	4.63	103	7.12
68	4.70	104	7.19
69	4.77	105	7.26
70	4.84	106	7.32
71	4.91	107	7.39
72	4.98	108	7.46
73	5.04	109	7.53
74	5.11	110	7.60
75	5.18	111	7.67

$$\text{FORMULA} = \frac{\text{VOL OF } H_2O \text{ IN C.C.}}{\text{AREA OF TUBE (SQ. CM.)}} \text{ X } \frac{\text{60 MIN}}{\text{COLLECTING TIME (30 MIN.)}} \text{ X } \frac{1}{2.54}$$

$$= \text{INCHES/HOUR}$$

PERCULATION TEST


WATER SUPPLY
WATER MUST BE KEPT AT THIS LEVEL
PERCULATION TUBE
FILL
BEAKER
NO. 100 MESH SCREENING
17"
15½"

TEST: Laboratory Percolation Rate Analysis

TEST LIMITATIONS: Four general test configurations have been devised to measure percolation rate behaviour of water through aggregate materials under laboratory observation. Several methods have been promoted for assessment of field percolation rate behaviour, though none are thought to be suitable for the purpose intended. Major problems associated with in-situ rate assessment include inability to approach suitable fill depths (at which representative samples exist) and the tendency to create disturbed (thus non-representative) samples upon installation of site equipment. For such reasons, percolation rate analyses, performed in the laboratory, rather than in-situ, settings, are normally preferred to characterize such behaviour.

TEST METHODS: Four standard test methods are normally utilized to perform permeability coefficient and percolation rate characterization of aggregates. The suitability of material percolation rate behaviour is based largely upon historical precedent - where material percolation rates exist at levels in excess of 10 cm/hour, the material is considered to offer adequate drainage potential and to be incapable of liquefaction failure following time consolidation.

METHOD A)- Constant Head Percolation Test

The apparatus for constant head analysis consists of a clear tube, at approximate internal diameter ranging between 3.5 - 10 cm, and of length between 10 - those, such as uncemented tailings, which exhibit unchanging percolation rate behaviour with time. Slurried tailings material is placed, at known density, within the tube assembly between two layers of porous stone and allowed to settle. Flow from the base of the installed tailings is permitted to occur and stabilize while a constant head of water is maintained above the top of the aggregate layer, as illustrated. Data which is recorded (flow distance between the two water sampling ports, the difference in water head between these two points, water temperature, elapsed sampling time and quantity of water percolating through the tailings mass during this interval) is used to determine the permeability coefficient for the material according to the following:

$$k = \frac{q \times L}{h \times A}$$

where: k = permeability coefficient, (m/hr)
q = rate of flow, (m^3/hr)
L = distance between tap points, (m)
h = static pressure differential across the porous medium, (m)
A = sample cross-sectional area, (m^2)

METHOD B) - Falling Head Percolation Test

Utilized for materials such as fine sands and clays which develop reduced permeability behaviour with time consolidation, the falling head test is used to characterize percolation rate behaviour of cement-consolidated tailings or of tailings which are consolidated as the result of tight confinement. Typical laboratory apparatus is illustrated and consists of a water standpipe, sample chamber into which tailings slurry is placed, top and bottom porous stone caps and various valve outlets. For testing, both top and bottom stopcock valves are opened and flow is permitted to occur through the sample into the constant head reservoir below.

Data which is recorded (initial distance between the standpipe meniscus and the constant reservoir surface, final distance between the standpipe meniscus and the constant reservoir surface, water temperature, elapsed sampling time, height of the test sample and cross-sectional area of the test sample) is used to determine the permeability coefficient for the material according to the following:

$$k = 2.3 \frac{(a \times l)}{(A \times t)} \log (h_o / h_1)$$

where:
k = permeability coefficient, (m/hr)
A = cross-sectional area of sample, (m^2)
a = cross-sectional area of standpipe, (m^2)
l = sample length through which flow occurs, (m)
t = duration of test, (hr)
h_0 = distance between initial standpipe meniscus position and lower reservoir surface level, (m)
h_1 = distance between final position of meniscus in standpipe and lower reservoir surface level , (m)

METHOD C) - Percolation Tube Test

The percolation tube test is a modified percolation rate measuring method which has been adopted for in-situ testing purposes. The apparatus, illustrated below, consists of two sections, generally plastic. The upper tube section exhibits an internal diameter of between 5 - 7 cm and a length of between 30 - 40 cm. Tubing is attached at the upper end to supply a constant head of water within the tube while a porous stone or mesh screen is fitted to the bottom to retain the emplaced fill sample. The lower tube section is a collection chamber into which percolating water will flow. Test performance is simple in execution - slurried tailings materials are poured into the upper tube section and a constant head of water is maintained on top of the sample by regulating water flow into the top of the upper tube section. Percolating water is collected within the lower chamber for fixed time intervals and pertinent test data (water temperature, sampling time interval, height of tailings within the percolation tube, height of water within the percolation column, cross-sectional area of the column and volume of water collected during the sampling interval) is utilized to determine the percolation rate according to the following:

$$K = \frac{q \times l}{h \times A}$$

where:
K = percolation rate, (cm/hr)
q = water flow rate through sample, (cm^3/hr)
l = height of tailings, (cm)
h = height of water within percolation column, (cm)
A = cross-sectional area of sample or flow column, (cm^2)

METHOD D) - Pressure Percolation Test

Developed by the Department of Civil Engineering, Queen's University, this elaborate apparatus is utilized to measure variation in percolation rate behaviour of tailings materials when subjected to variable loading stresses similar to those anticipated to develop during typical fill installation cycles. The typical apparatus is illustrated. Tailings samples are placed between porous surfaces within a test cylinder which is attached to a collection reservoir below.

A top loading platen, through which supply water is passed, is placed in contact with the upper sample surface. Percolating water volumes are collected during measured time intervals. After each collection stage, incremental specimen loads are applied and timed water collection is repeated. Tests can be conducted using both non-cemented and cemented tailings materials, and, when fully drained and time cured samples are prepared, the apparatus can be used to perform unconfined compression failure tests as well. Percolation rate analysis is performed in a manner similar to that previousaly discussed for the percolation tube test. Due to the high cost of the pressure percolation apparatus (estimated at $2000), relative to that of the other techniques described, and to the equipment complexity necessary for sample loading, the use of such a test would be prohibitive for most mining operations.

Results obtained from the pressure percolation test are derived in a manner similar to that which is performed for the percolation tube method, previously described.

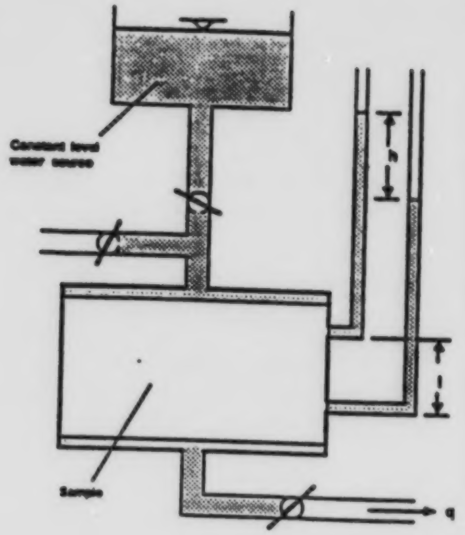

Constant level water source
h
l
Sample
q

FIGURE A-1 - Constant Head Laboratory Permeability Test

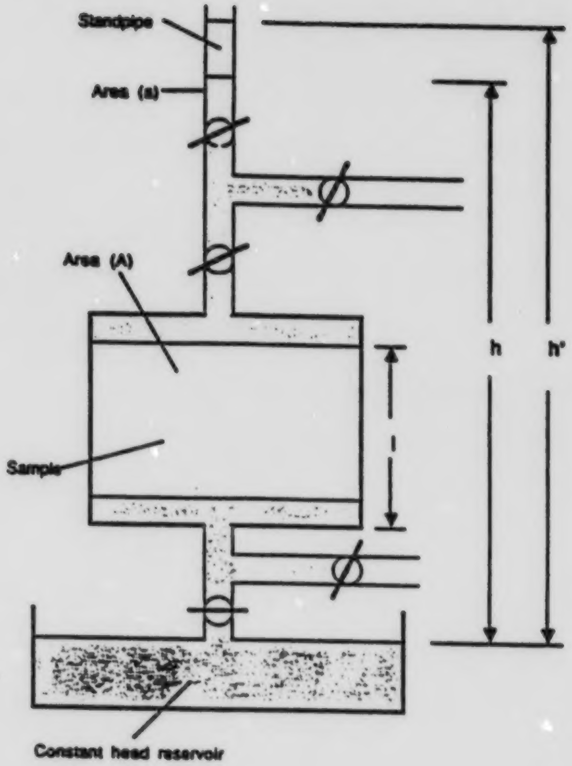

Standpipe
Area (a)
Area (A)
Sample
l
h
h'
Constant head reservoir

FIGURE A-2 - Falling Head Laboratory Permeability Test

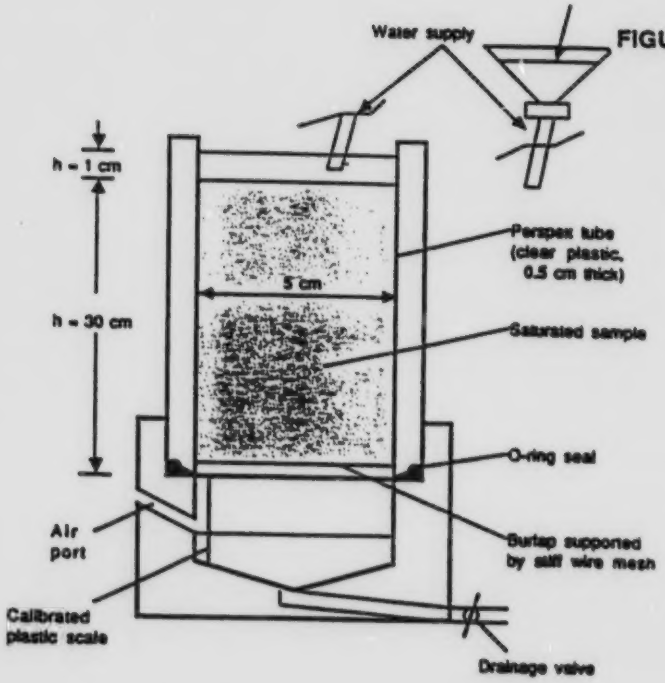

Water supply
h - 1 cm
h - 30 cm
5 cm
Perspex tube (clear plastic, 0.5 cm thick)
Saturated sample
O-ring seal
Burlap supported by stiff wire mesh
Air port
Calibrated plastic scale
Drainage valve

FIGURE A-4 - Percolation Test Arrangement

TEST: Laboratory Density Analysis (bulk, or apparent, and true solids density)

EQUIPMENT NEEDS:

- Pycnometer (mercury/air type or inert gas/air type)
- Graduated cylinders or flasks
- Electronic or mechanical balance having weight sensitivity to ± 0.01 g.
- Estimated equipment costs - $2000 Cdn

TEST METHOD: A) True solids density (or specific gravity) of aggregate materials can be determined using one of two general methods, these being by the water immersion technique or through the use of pycnometers.

Pycnometer Operation

- Considered to be more reliable and accurate than the manual method of water immersion, pycnometers are suitable for use with both dry and moist solids or powder materials.
- In the mercury/air type, known volumes and weights of mercury are used to calibrate the equipment; in the inert gas types, partial pressure differentials of non-absorbing gases are utilized to determine accurate volume measurements of known weights of placed materials.
- Mercury pycnometers are labour intensive, slow to use, may promote mercury contamination of work spaces and provide standard accuracy of ± 0.05 g/cm^3 though they are cheap to manufacture and can be made in-house.
- Inert gas pycnometers are generally of the "black box" type, in that users must follow a well defined series of programmed steps to achieve volume measurements; manually-operated instruments provide readings in approximately one tenth the time at accuracies of ± 0.02 g/cm^3 though costs of several thousands of dollars per instrument must be budgeted.
- Both systems are calibrated using known volume samples to create differential pressure effects in a fixed volume sampling chamber; when a solid of unknown volume is placed within the chamber a pressure difference is created which can be scaled against known volume standards.
- Inert gas pycnometers operate using differential pressures developed between a reference gas pressure and atmospheric pressure; mercury pycnometers are referenced between atmospheric pressure and that developed by a sealed air source.
- The specific methodology varies with the type of instrument used; detail concerning the use of each type is included in the summary report.

Water Immersion Technique

- Best suited for coarse, non-hydrophobic materials, the method requires that a filled flask (of known weight), containing water at a specific temperature, be filled to a set level and accurately weighed.
- The dry flask and known weight of sample are then weighed, and a known volume (and mass) of water is used to fill the flask.
- The true solids density of the solid is determined according to the following:

$$\text{Volume sample } (V_s) + \text{Volume water } (V_w) = \text{Volume flask } (V_f)$$

$$\frac{\text{Sample mass (dry)}}{\text{Solids density}} + \frac{\text{Water mass}}{\text{Water density}} = \text{Volume flask } (V_f)$$

B) The bulk density, or apparent bulk density, of placed materials is determined by weighing the total mass of a typical aggregate sample and dividing this by the total sample volume (volume of solids, voids and liquids). Unlike true solids density (specific gravity) values, these can be affected by the degree of consolidation, changing moisture contents and the like.

Method of Determination

- a known sample weight (plus that of the container if the sample is unconsolidated) is recorded fcr measured moisture content conditions
- if the sample geometry is regular, the sample volume can be determined based upon surface measurements; if irregular in shape, the sample can be thinly coated in wax (to seal the sample pores) and immersed in water to determine the volume displaced.
- where cohesionless (and usually dry) aggregate materials are to be tested, the "free poured" density is derived
- for such materials, a weighed, graduated cylinder is filled with a known weight of sample
- the volume of the cylinder which is occupied by the known mass of aggregate is determined, and the apparent bulk density of material is calculated by dividing the total sample weight by the apparent volume occupied.

Physical Parameters Derived

Determination of the true solids and apparent bulk density parameters of a tailings material can be utilized to assess two additional parameters, these being the "Void Ratio" and the "Porosity".

The void ratio (e) is a measure of the ratio of the sample voids volume to the sample solids volume; the solids volume, (V_s), is that which is measured by pycnometer, and the voids volume, (V_v), is the total apparent sample volume less the volume of solids in that sample. Thus:

$$e = (V_v)/(V_s)$$

The sample porosity , (P), is the measure of the ratio of the volume of voids contained in the sample to the entire sample volume (voids plus solids), and can be represented by:

$$P = \frac{(V_v)}{(V_v) + (V_s)}$$

TEST: Laboratory Sieve Size Analysis

EQUIPMENT NEEDS:
- Tyler sieve screens of appropriate mesh sizes
- Sieve shaker apparatus
- Sieve cleaning accessories
- Weighing balance having peak capacity of 300 g ± 0.01g

GENERAL:

Sieve sizing analysis is one of the necessary and fundamental characterization tests which must always be performed on backfill materials (with the possible exception of rock fill materials). Although six closely related standard sieve classifications are in use in the Western world, the Tyler series is very widely adopted and is recommended for use by all operators in order to avoid confusion in reporting and comparing size data.

Standard sieve equipment, unlike certain of the load frame assemblies which are described for performance of strength analyses, are difficult to manufacture in-house and must be purchased commercially. Because such apparatus is commonly utilized in mill operations, neither access to nor the cost of such equipment for backfill testing purposes should be a limiting factor in adopting such tests by mine operators for quality control examinations. The process of sieve sizing combines many inherent problems which are remedied by adhering to standard operating practice. In order to maintain a ~~brief operating~~ summary, some of the important sieving procedures which should be followed to insure that quality sizing information is obtained will be communicated.

TEST METHOD:

1) Large sample weights should always be split by means of wetting the material to insure that no fines are lost by agglomeration
2) Screens and equipment should always be inspected before use to check for screen damage which could result in loss of material
3) Special attention must be paid to the - 40 μm fines fractions as aggregate materials at this size range have the most significant effect on backfill strength and permeability rate
4) As coarse materials are commonly sieved dry, establish standard drying practice; typical drying temperatures should not exceed 30-40^0 C, otherwise reactive tailings component materials (such as sulphides) will be physically altered
5) Typical batch sizes of tailings aggregate to be sieved should approximate 200 g of dry solids existing at ambient temperature conditions; screening of quantities greater than this often will result in blinding of the screens and surface damage due to excess weights carried.
6) Batch quantities should be initially wet screened through a 53 μm (270 Tyler mesh) sieve to remove the sub-sieve size fractions where cyclosizer apparatus is locally available to permit accurate size analysis of this fines fraction material; where cyclosizers are not available, the process of wet sieving serves no benefit in sizing
7) Where sub-sieve wet screening is carried out, cyclosizer analysis at 40, 30, 20, 15 and 10 micron sizes is performed
8) Where cyclosizer analysis is conducted, the wet oversize material is again dried and will be subjected to dry sieve analysis; where no cyclosizer analysis is feasible, the entire tailings product (fines and coarse) will be subject to dry sieving analysis
9) Dry sieving using the following Tyler series mesh screen sizes is normally performed: 8,10, 14, 20, 28, 35, 48, 65, 100, 150, 200, 270, 325 and 400. Where the tailings contains few of the coarser fractions, some of the coarser screens may be deleted .

10) If the tailings consists of a high proportion of fines, overloading or blinding of some of the finer sieves will result; it is then advisable to reduce the overall sample weight below the suggested 200 g batch limit to prevent this condition.

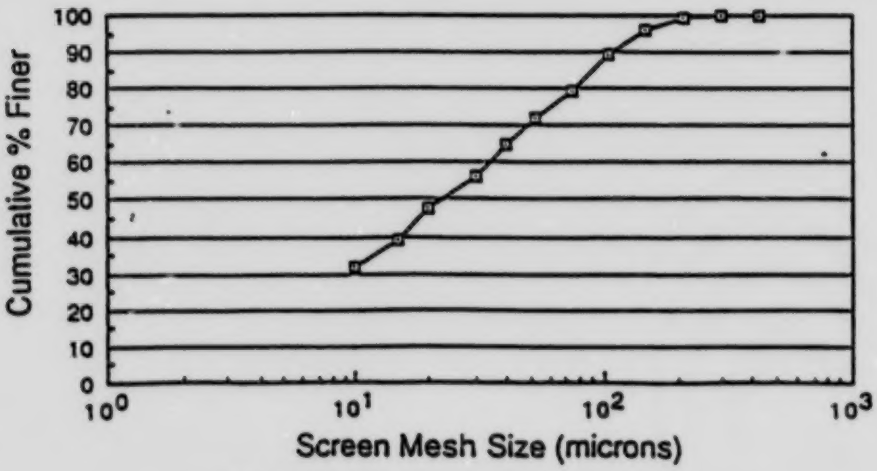
11) As particle sizing takes place gradually, complete batch separation by screening may take up to 60 minutes to accomplish. In order to assure that reproducible sieving takes place between successive batch samples, a recommended sieve shaking period of 60 minutes should be set.

12) Following shaking, the material on each screen is removed and weighed.

13) The respective screen weights are summed, expressed as a percentage of the total original material weight and plotted in one of several fashions to illustrate the material size distribution data; it is common to plot such results using a cumulative percent finer (or percent passing) versus mesh (or opening diameter) size relationship as a log-normal distribution. Typical sieve size data and distribution plots are illustrated.

Primary information which can be obtained from tailings size distribution dat~~a is~~ the Coefficient of Uniformity (C_u) parameter, empirically derived as the ratio of the particle diameter at which 60% of the material (by weight) is passed versus the diameter of the 10% passing size. The Coefficient provides a qualitative measure of the density and strength character of non-consolidated tailings materials; at values of $(C_u) = 5.0$, low porosity and optimum fill density conditions are inferred to exist. Alternately, where $(C_u) = 1.0$, the tailings product is indicated to consist of uniformly-sized material and to exhibit both high porosity and low strength capability.

	Tyler Screen	Aperture (µm)	Wt. Retained	% Retained	Cum. % Ret.	Cum. % Finer
1	3	6700	0.0	0.0	0.0	100.0
2	4	4750	0.0	0.0	0.0	100.0
3	6	3350	0.0	0.0	0.0	100.0
4	8	2360	0.0	0.0	0.0	100.0
5	10	1700	0.0	0.0	0.0	100.0
6	14	1180	0.0	0.0	0.0	100.0
7	20	850	0.0	0.0	0.0	100.0
8	28	600	0.0	0.0	0.0	100.0
9	35	425	0.0	0.0	0.0	100.0
10	48	300	0.2	0.1	0.1	99.9
11	65	210	1.58	0.8	0.9	99.1
12	100	150	6.12	3.1	4.0	96.0
13	150	106	13.03	6.6	10.6	89.4
14	200	75	19.93	10.1	20.8	79.3
15	270	53	14.21	7.2	28.0	72.1
16	325	45	-----	-----	-----	-----
17	400	38	-----	-----	-----	-----
18						
19	TOTAL	---	-----	-----	-----	-----
20						
21	1	40	14.21	7.2	35.2	64.9
22	2	30	17.17	8.7	43.9	56.2
23	3	20	16.38	8.3	52.2	47.9
24	4	15	16.77	8.5	60.7	39.4
25	5	10	14.80	7.5	68.2	31.9
26	PAN	-10	63.13	32.0	100.0	0.0
27						
28	TOTAL	---	197.55	-----	-----	-----
29						

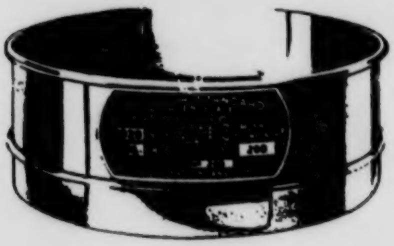


TAILINGS SIZE DISTRIBUTION CURVE

HOW TO ORDER SIEVES

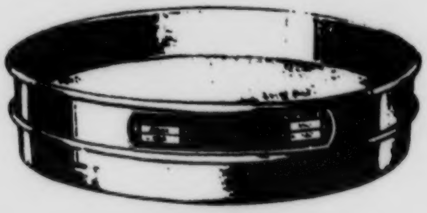
SPECIFY—Diameter Desired - Whether Half or Full Height - U. S. Series or Tyler Standard—
Sieve Designation - Brass or Stainless Frame - Brass or Stainless Cloth.

DIMENSIONS OF STANDARD 8" DIAMETER TESTING SIEVES



Full Height

Height 2⅞", depth to cloth 2"



Half Height

Height 1⅞", depth to cloth 1"

Dimensions of Other Sizes Available

Full Height

3" Dia., height 1¾", depth to cloth 1¼"
6" Dia., height 2⅜", depth to cloth 1¾"
10" Dia., height 4", depth to cloth 3"
12" Dia., height 4¼", depth to cloth 3¼"

Half Height

3" Dia., height 1¼", depth to cloth ⅝"
6" Dia., height 1⅞", depth to cloth 1"
10" Dia., height 2½", depth to cloth 1½"
12" Dia., height 2⅝", depth to cloth 1⅝"

PANS AND COVERS

Bottom Pan—For use at bottom of nest.

Pan with Extended Rim—For nesting in a series of sieves, the same as the sieves themselves.

Flat Cover or Flat Cover with Recessed Wire Ring Handle.

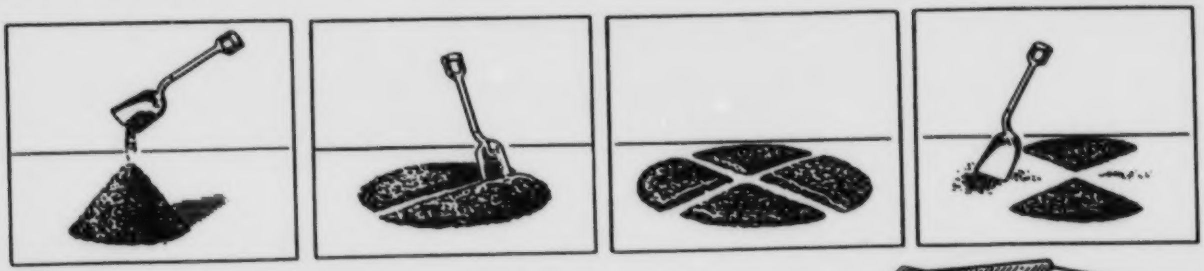
SUGGESTIONS FOR TESTING

How to "take" samples

Great care must be taken to secure samples that are truly representative. One shovelful of material taken at random will not be representative. At the apex of a conical pile the proportion of fines is excessive. At the base the proportion of coarse materials is excessive.

In the shoveling process every fifth shovel, or every tenth shovel, etc., should be taken depending upon the amount of sample desired. The sample should consist of small quantities taken at random from as many parts of the pile as are accessible, in such a manner that the proportion of coarse and fines in the small amount will be the same as in the larger amount.

Greater accuracy in the sampling can be obtained where material is flowing in a chute or launder (either with or without water). The ideal place to take the sample is just where the material drops from the chute. When taking the samples, use a pail or a specially built receptacle which can be swung completely across the flowing stream in a brief interval of time, so as to take "all of the stream part



of the time." Under no circumstances should the sampling receptacle be allowed to overflow.

Coning and quartering

The sample should be piled in a cone, each shovelful going to the center of the cone and allowed to run down equally in all directions—this will mix the sample. Then spread out in a circle and walk around the pile gradually widening the circle with a shovel until the material is spread out to a uniform thickness.

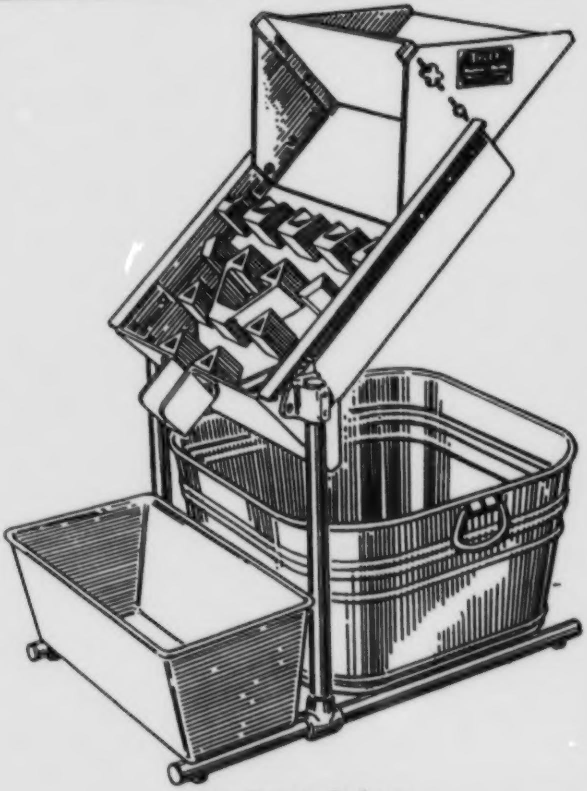
Mark the flat pile into quarters, and reject two opposite quarters. Mix again by shoveling the material into a conical pile, taking alternate shovelfuls from the two quarters saved. Continue the process of piling, flattening, and rejecting two quarters until the sample is reduced to the required size.

Sample Splitters

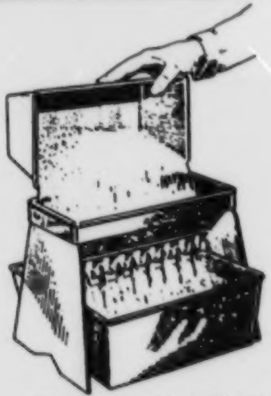
Samples taken with mechanical sampling devices are more reliable than samples taken by hand. The Tyler 16-1 Sample Reducer is ideal for this purpose. This reducer will cut out a representative 1/16th part of the material fed to it. With two units in tandem or passing the product over the same unit a second time, a one pound sample can be obtained from an original 256 pound sample. Three passes through this splitter will give you a representative one pound sample from two tons of material.

To further reduce the sample to obtain the right amount for the test the Tyler Sample Splitter should be used. A sample splitter should be an essential part of every laboratory. Sample Splitters divide a sample in half, but by repeated passes the sample can be split into quarters, eighths, etc.

Do not attempt to arrive at exactly the amount of material specified for the test. If a 50 gram sample is to be used in the test, get as near to this amount as you can by passes through the splitter as a 49 or 51 gram sample will make no difference



16-1 Sample Reducer



CL-280 Sample Splitter

SUGGESTIONS FOR TESTING (cont.)

in the test. If you try to obtain exactly a 50 gram sample, the tendency is to spoon out a little material from a slightly larger sample. This should never be done as spooning will not remove a representative part.

Surveys have shown that the greatest cause of inconsistencies in testing results have come from improper selection of the sample which is not truly representative of the entire lot of material. Therefore, every care should be taken in the proper selection of the sample. Once a sampling procedure is established that will give consistent results, this procedure should be carefully followed in every test.

PROCEDURE IN MAKING SIEVE ANALYSES

In establishing a procedure for standardization of sieve analyses we must consider the size or weight of sample to be tested, the length of sieving time, required accuracy of test, and the type of test—wet or dry.

Size of sample to be tested

In determining the size or weight of a sample we must consider the type of material, its screenability, and the range of particle sizes present. For example, in making a sieve analysis of a material representing a feed to a screen or a product from a crusher in which the particle range is very wide, a large sample of from 500 up to 1,000 grams may be required. If the material to be tested is a finely ground product, a sample of 25 to 100 grams should be used.

There is a natural tendency to use too large a sample in the test. This is incorrect as in general the smaller the sample (properly taken), the more consistent the results. In order to obtain an accurate sieve test every particle must present itself to the screen openings for rejection or for passing through to the next finer sieve. If there are six or seven sieves in the nest, a fine mesh particle must repeat this operation six or seven times. If the sieves are overloaded, the fine mesh particle may never get a chance to get to its proper sieve. However, the sample should be large enough so that the first sieve retains enough particles to be representative.

The general rule in determining the size of a sample is that it be limited in weight so that no sieve in the series used in the analysis be overloaded. Overloading is most likely to occur in making analyses on closely graded materials where the range of particle size is confined to close limits. In this case the size of sample should be determined by the capacity, without overloading, of the sieve retaining the largest amount of the sample. Overloading of the sieves results in unreliable data as blinding of the meshes occurs on the heavily loaded sieve.

As an aid in determining the size of the sample, the following procedure is suggested:

Accurately split out on a sample splitter samples of varying weights—as for example, 25, 50, 100, 150 and 200 grams of the material. Then run these various samples on the sieves selected for a period of say five minutes. A comparison of these results will definitely show the correct size of sample to use.

For example, if the 100 gram sample shows approximately the same results in percentages retained and passing the sieves as the 50 gram sample, whereas the 150 gram sample shows less material through the finest sieve, this would be an indication that a 100 gram sample would be satisfactory for tests.

Near-mesh particles are those having dimensions which are close to the sieve opening and in order to secure analyses which are accurate, it is essential that the sieves be lightly loaded so that each of these near-mesh particles can be presented to the sieve opening many times and thus allow maximum opportunity for accurate classification.

Testing Sieves

For most sieve analysis tests the standard 8" diameter brass frame sieves should be used. The full height size (2" deep) is standard but if desired half height (1" deep) sieves are also available.

If possible full height sieves are recommended as the operator has to be very careful with half height sieves in the weighing operation as they are very susceptible to spilling and ruining the test.

The sieves are designed to nest together so that when all of the sieves used in the test are assembled together, a solid stack of sieves is obtained ready for hand sieving or mechanical sieving in the Ro-Tap Testing Sieve Shaker (CL-340).

Ro-Tap testing sieve shaker (CL-340)

The Ro-Tap Testing Sieve Shaker reproduces the circular and tapping motion given testing sieves in hand sieving but with a uniform, mechanical action, producing dependable sizing tests.

The superiority of mechanical sieving, as performed by the Ro-Tap is plainly indicated by the

SUGGESTIONS FOR TESTING (cont.)

results of the test which follows. A sample of finely ground low-grade copper ore was used. The material was free from lumps and contained much material which would ordinarily be referred to as "slime," "flour," or "dust." A 100 gram sample was obtained by means of a sample splitter.

Three tests of twenty minutes each were made with two Ro-Taps, the same sample being used for all three tests. Test No. 1 was run on Ro-Tap "A", Test No. 2 was a repetition of Test No. 1. Test No. 3 was a repetition of Test No. 1 but run on Ro-Tap "B". The result of these tests show that data obtained with Ro-Taps is comparable:

Opg. Micron	U.S. No.	Tyler Mesh	Test No. 1 Ro-Tap "A" Per Cent Weight	Test No. 2 Ro-Tap "A" Per Cent Weight	Test No. 3 Ro-Tap "B" Per Cent Weight
595	30	28	33.4	33.5	33.7
420	40	35	18.1	18.0	18.0
297	50	48	11.0	11.1	11.1
210	70	65	8.2	8.2	8.2
149	100	100	7.2	7.3	7.2
74	200	200	9.3	9.4	9.4
	Pan	Pan	12.8	12.5	12.4

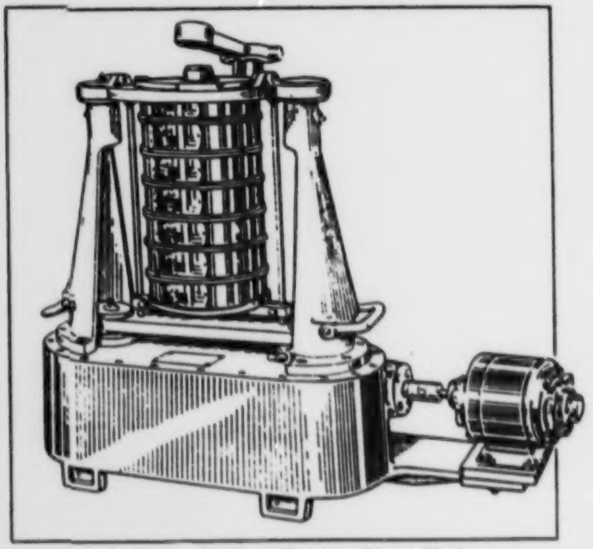
It will be seen that the results obtained in the three tests are in substantial agreement. A comparison of Test No. 1 with Test No. 2 shows that the machine will repeat results on the same sample, and a comparison of Test No. 1 and No. 2 with Test No. 3 shows that the sieving action of any two machines is practically identical.

With the regular height sieves, one sample can be tested on a series of six sieves of different openings, while, with the half height sieves, one sample can be analyzed on a series of thirteen sieves, and all with one operation.

By using pans with nesting skirts three different samples can be tested at one time with the regular height sieves, and seven different samples with the half-height sieves.

Length of sieving time

The time required for sieving in the Ro-Tap Testing Sieve Shaker is dependent upon the type of test desired. For example, in many instances for plant control operations a 3 to 5 minute test on a free sieving material is sufficient to give the desired data; whereas, on more difficult materials sieving time of from 10 to 30 minutes is justified.



CL-340 Ro-Tap Testing Sieve Shaker

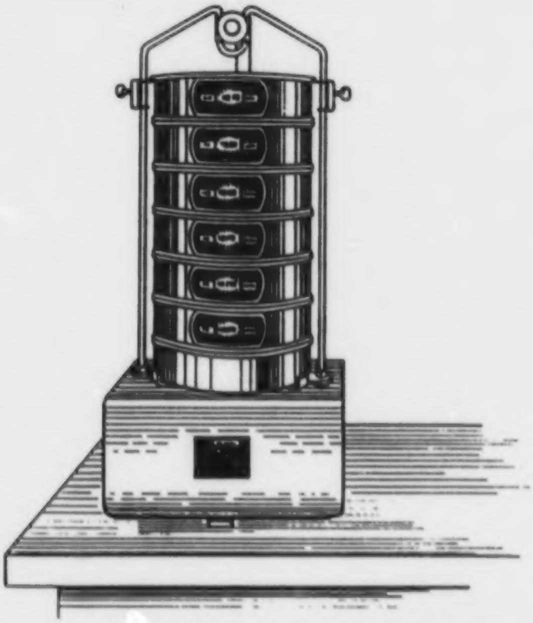
If sieve tests are made for the purpose of determining whether or not the material meets with definite specifications, a longer period of sieving may be established. All interested parties, however, should agree and follow a standardized method as in only this way will their tests be comparable.

In determining the length of sieving time necessary it is suggested that three or four samples be cut out on a sample splitter to the weight which has been previously proven as satisfactory. One of these samples could then be sieved for a 5-minute period, one for 10 minutes, another for 15 and a fourth for a 20-minute sieving period. After tabulating these various results by percentages, the length of time necessary to stabilize the sieving action will be readily apparent, or in other words, a practical "end point" of sieving can be determined. A satisfactory "end point" is considered to have been reached when an additional period of sieving time fails to change the results on any of the sieves used in the analysis by more than 0.5% to 1.0%. In reporting sieve tests, calculations carried to 0.1% are the limit of accuracy justified except in very unusual cases.

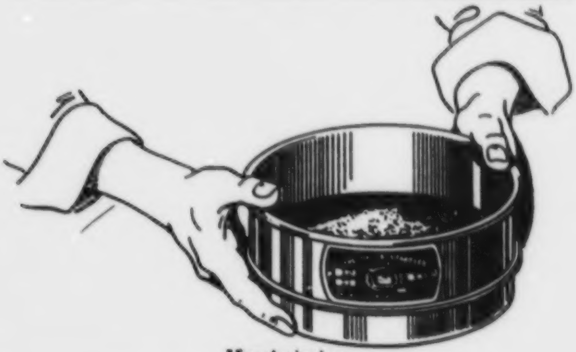
Tyler Portable Sieve Shakers

There are many testing operations that do not require the accuracy and consistency of results as obtained with the Ro-Tap. Also in some cases the number of tests made do not warrant the cost of the Ro-Tap. For these purposes the Tyler Portable Sieve Shaker is available in both a hand operated, gasoline motor, or electric motor driven design.

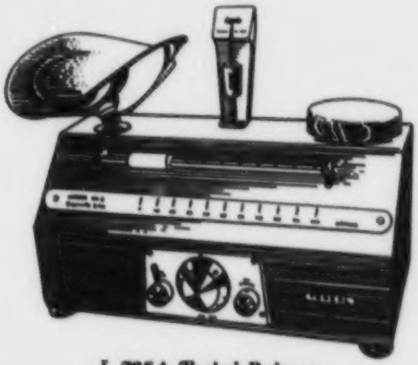
SUGGESTIONS FOR TESTING (cont.)



Motor-Driven Portable Sieve Shaker



Hand sieving



L-725A Torbal Balance

These Portable Sieve Shakers are the most inexpensive units on the market and while they do not give the accurate and consistent results as obtained on the Ro-Tap, they may be satisfactory for routine field testing.

Hand sieving

The Ro-Tap Sieve Shaker has become a universally accepted method of making sieve analysis. However, for those who might wish to make an occasional hand test the following suggestions are offered. In hand sieving, arrange the various sieves to be used in one test, the coarsest opening at the top, and place the sample to be sieved in the top sieve. The whole nest of sieves should then be given a preliminary shaking for two or three minutes. After this preliminary shaking, each sieve (starting with the coarsest) should be shaken separately to complete the separation. The time required for shaking the sieves will vary with the fineness of the sieve and the character of the material that is to be sieved. Where tests are made for the purpose of comparison, a standard procedure should be adopted.

Weighing the Sample

After completion of the agitation of the sieves the entire nest of sieves should be brought to the weighing station for recording of the analysis. Paper for recording the analysis should be available similar to Form L-1 as shown on Page 10.

Weighing should always be done by grams and a balance having at least a capacity of 500 grams with a sensitivity of 1/10 gram is desirable. The Model L-725A Torbal Balance shown here.

Using a spare bottom pan the material retained on the coarsest sieve should be dumped into the pan and the sieve inverted and placed over the pan. Then a soft brass wire brush or nylon bristle brush as shown on Page 9 is used to gently brush the underside of the sieve using a circular motion, being careful not to exert too much pressure against the wire cloth.

In most every case virtually all the near mesh particles imbedded in the meshes can be removed by this dry brushing process. The sieve can then be raised from the pan and the side of the frame tapped by the handle of the brush to clean the remaining

SUGGESTIONS FOR TESTING (cont.)

material on the sieve. Weigh the contents in the pan generally to the nearest 1/10th gram and this figure immediately recorded.

If several extra pans are available it is best not to discard this portion of the sample until the entire weighing is completed. This same procedure should be repeated on all the sieves in the nest. The material passing through the finest sieve into the bottom pan must also be weighed to obtain total weight for percentage calculations and to permit a check against the original weight of sample. The total weight of the material retained on the various sieves should be very close to the weight of the original sample.

The percent retained on each sieve is calculated merely by dividing the weight of the material retained on a particular sieve by the weight of the original sample. The cumulative weight retained on the sieve and all coarser to it should also be calculated and entered in the proper column on the tabulating paper.

Most industries set up their specifications by the percent of material retained on a particular sieve, however, in some industries the percent passing a particular sieve is used.

The weight of the sample or the weight of the material retained on each sieve is never used for comparative purposes, but all results are expressed by the percentage of the total sample retained or passed through a particular sieve.

Wet Testing

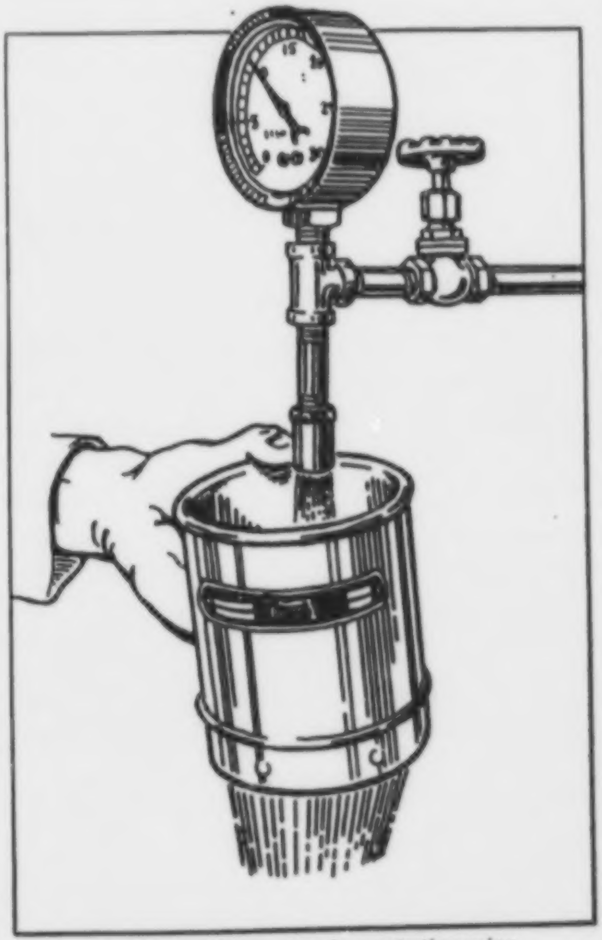
If difficulty is encountered in obtaining reproducible results on materials difficult to screen, and if the material is not soluble in water,very accurate sieve analysis tests can be made by the wet testing method.

If the material readily mixes with water, first place the material in the desired sieve and then wash the material through the sieve using a rubber hose connected to a water tap, pressing the rubber hose together to give a flat stream of high pressure but low volume. Care should be exercised when first applying the water to prevent any dry material spilling over the frame of the sieve.

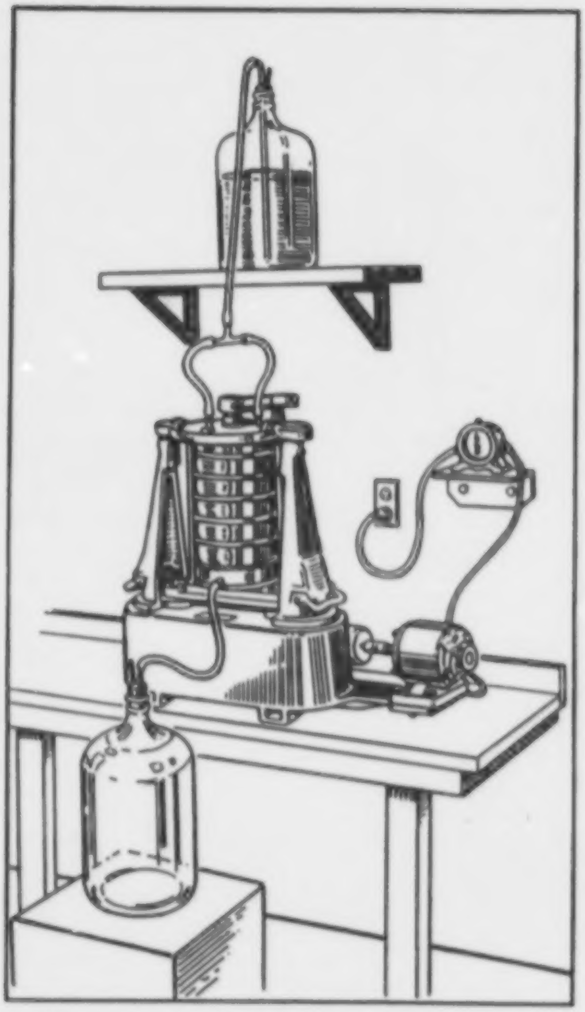
Special deep frame sieves designed for wet testing are available for this purpose (See Page 7). A water tap fitted with a pressure gauge and spray nozzle can also be used as shown in the illustration.

If the material does not mix well with water, first place the accurately weighed sample of dry material in a quart jar and fill the jar about three quarters full of water. The contents should be shaken vigorously to thoroughly mix the material with the water. This mixture can then be dumped into the sieves and the washing process continued as described above.

The washing should continue until clear water passes through the sieve. The sample is then dried and the residue loosened by brushing to eliminate any caking. The residue is then weighed and percentages calculated from the original total weight of the sample. The amount passing through the sieve is assumed to be the difference between the original sample weight and weight of the residue on the sieve.



CL-386 Deep frame sieve for wet testing using CL-364 Wet washing attachment



Wet testing with the Ro-Tap Testing Sieve Shaker.

Drying the Sample

The entire sieve containing the wet material should be dried in an oven if possible, being sure that the temperature does not exceed 300° F. or 150° C. Damage will occur to the sieve if temperatures in excess of this are used.

If a drying oven is not available, rig up a rack over an open gas flame so that the sieve can be suspended about 18 inches to 24 inches above the gas flame. This will provide sufficient heat to dry the material and still not damage the sieve.

Wet Testing Using The Ro-Tap Testing Sieve Shaker

The advantages of mechanical sieving can be used in wet testing by the procedure described below.

For wet tests, all that is required is to equip a Ro-Tap Testing Sieve Shaker with a special cover with inlet pipes and a special bottom pan with a drain pipe as shown in the illustration at the left.

Water is then introduced into the nest of sieves while they are being shaken, and this washing action in many cases aids in the sieving result. It is necessary to dry each sieve and weigh the residue.

If interested in more specific details on this method of wet testing, please contact Soiltest, Inc.

Combined Wet and Dry Testing

In many cases when an analysis is to be made on a nest of sieves, it cannot be satisfactorily done on a dry basis due to the presence of extremely fine particles which either agglomerate or cause blinding on the coarser sieves. It is found that by first removing the extremely fine particles from the sample, a very satisfactory test can be made on a dry basis.

To achieve this, a combined wet and dry method can be used. The specified weight of sample is first placed in the finest sieve in the nest. The material is then washed through the sieve using the process as described on Page 7.

The residue remaining on the finest sieve is then dried, and the test is conducted in the Ro-Tap Testing Sieve Shaker on a dry basis using all the sieves in the nest including the finest sieve on which the material was washed. The difference between the original weight of the sample and the total weight retained on the various sieves used represents the amount passing the finest sieve.

In any wet test, since the amount of material passing through the finest sieve cannot be weighed, great care must be exercised in weighing the material retained on the other sieves, as totals cannot be checked.

Care and Cleaning of Testing Sieves

A Testing Sieve is a scientific precision instrument and should be treated accordingly. Care should be exercised in the handling of the sieves so as not to damage the wire cloth or frame.

SUGGESTIONS FOR TESTING (cont.)

After each sieve analysis test the sieves should be carefully cleaned and stored in a cabinet or in their original cartons. [redacted] should be used for cleaning the sieves, and this is best done by brushing the underside of the wire cloth with a circular motion taking care not to exert too much pressure against the wire cloth. The frame of the sieve may be gently tapped with the wooden handle of the brush.

Occasionally it may be necessary to wash the sieves in a warm soap and water solution to remove the near mesh particles. The underside of the sieve can be brushed while in the water to aid in the cleaning action.

[redacted] for cleaning sieves, as the acid will reduce the diameter of the wire and enlarge the openings. It will also make the wire cloth sleazy.

If the above methods fail, the safest method to use is to dip the sieves in a 5% solution of boiling hot acetic acid, and then after brushing the particles free from the openings the sieves should be very thoroughly washed in water to remove all traces of the acid.

Experience has shown that if the sieves are carefully cleaned with the brass wire brush after each sieve test and possibly occasionally washed in soap and water, no serious blinding of the wire cloth should occur.

If properly handled and cared for, testing sieves should retain their accuracy during the life of the sieve, since ordinary wear on the wire screen has no measurable effect on the size of the opening. Most of the abrasive action is on the top of the wire knuckles and so does not affect the opening.

Cleaning sieve with wire brush

Tyler Testing and Research Laboratory

Special Testing Sieve Problems

Should your laboratory have difficulty in obtaining consistent sieving results or require assistance in other ways, [redacted] The Tyler Testing Laboratory is particu- [redacted] The Tyler Laboratory has played a large part in establishing standards on virtually all types of materials.

The Tyler Laboratory can be of particular help in determining the cause for discrepancies in results. They can make suggestions for improving your sieving procedure, and the laboratory is also equipped to furnish matched sieves for greater accuracy.

If you have a testing sieve problem of this kind, contact Soiltest, Inc. and they will see that it is given full consideration.

HOW TO RECORD AND COMPARE SIEVE ANALYSES

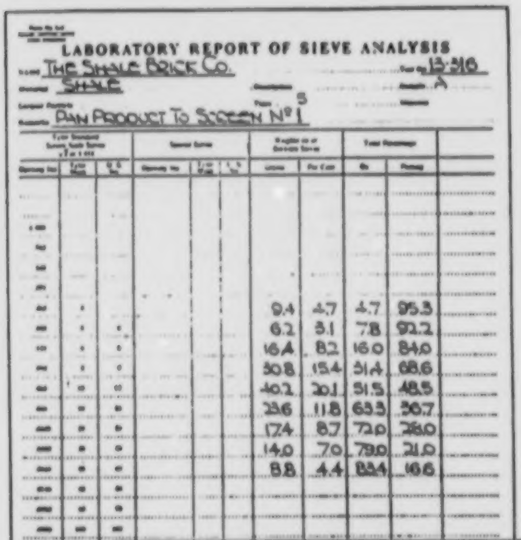
On this page are shown two forms which have proved particularly useful for recording and comparing the results of sieve analyses.

Form L1 is intended for use in recording weights obtained when making a sieve analysis and for recording percentages calculated from these weights. The illustration of this form shows a complete sieve analysis, including total or cumulative weights.

On Form L3, several analyses may be entered for comparison. In the illustration, actual analyses used in studying improvements in screening processes in a shale brick plant were entered in the proper columns. The analyses were first recorded on Form L1 and then entered on Form L3.

Plotting curves

There are many advantages in the graphic method of showing the data obtained in a screen analysis. Plotted curves of crushing results by different methods or from two competing machines express the differences in a more striking and concise manner than does tabulated data. Of the several methods of plotting these curves, the cumulative direct plot and cumulative logarithmic plot are the two most valuable and generally used. The curves in either plan are plotted by marking the cumulative* percentages of the material which remain on the sieves



LABORATORY REPORT OF SIEVE ANALYSIS

THE SHALE BRICK CO.

13-316

SHALE

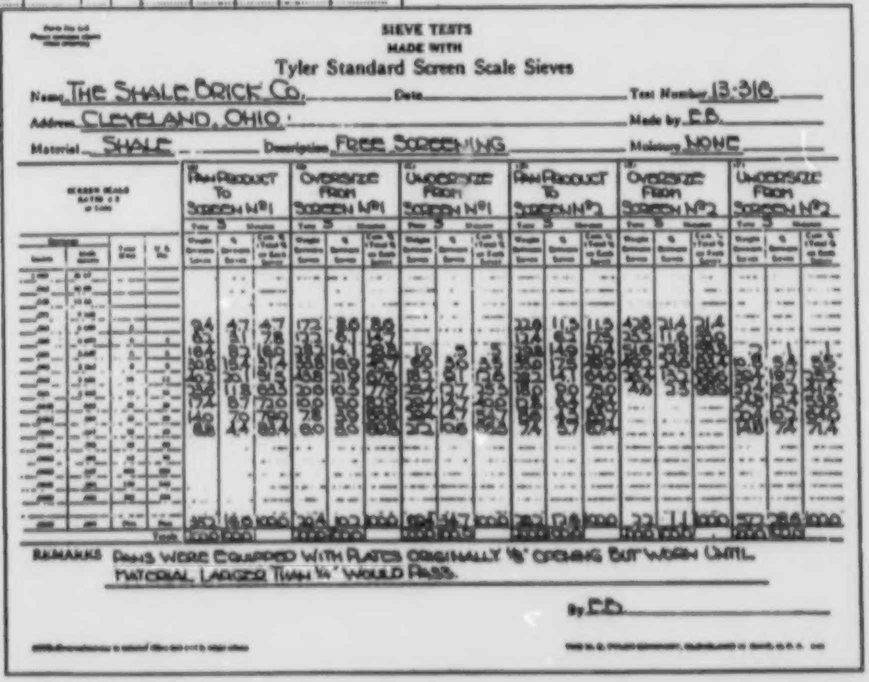
A

5

PAN PRODUCT TO SCREEN No 1

9.4	4.7	4.7	95.3
6.2	3.1	7.8	92.2
16.4	8.2	16.0	84.0
30.8	15.4	31.4	68.6
40.2	20.1	51.5	48.5
23.6	11.8	63.3	36.7
17.4	8.7	72.0	28.0
14.0	7.0	79.0	21.0
8.8	4.4	83.4	16.6

Form No. L-1



SIEVE TESTS
MADE WITH
Tyler Standard Screen Scale Sieves

Name THE SHALE BRICK CO. Date Test Number 13-316

Address CLEVELAND, OHIO. Made by E.B.

Material SHALE Description FREE SCREENING Moisture NONE

PAN PRODUCT TO SCREEN No 1	OVERSIZE FROM SCREEN No 1	UNDERSIZE FROM SCREEN No 1	PAN PRODUCT TO SCREEN No 2	OVERSIZE FROM SCREEN No 2	UNDERSIZE FROM SCREEN No 2
Time 5	Time 5	Time 5	Time 5	Time 5	Time 5

REMARKS PANS WERE EQUIPPED WITH PLATES ORIGINALLY ⅛" OPENING BUT WORN UNTIL MATERIAL LARGER THAN ¼" WOULD PASS.

By E.B.

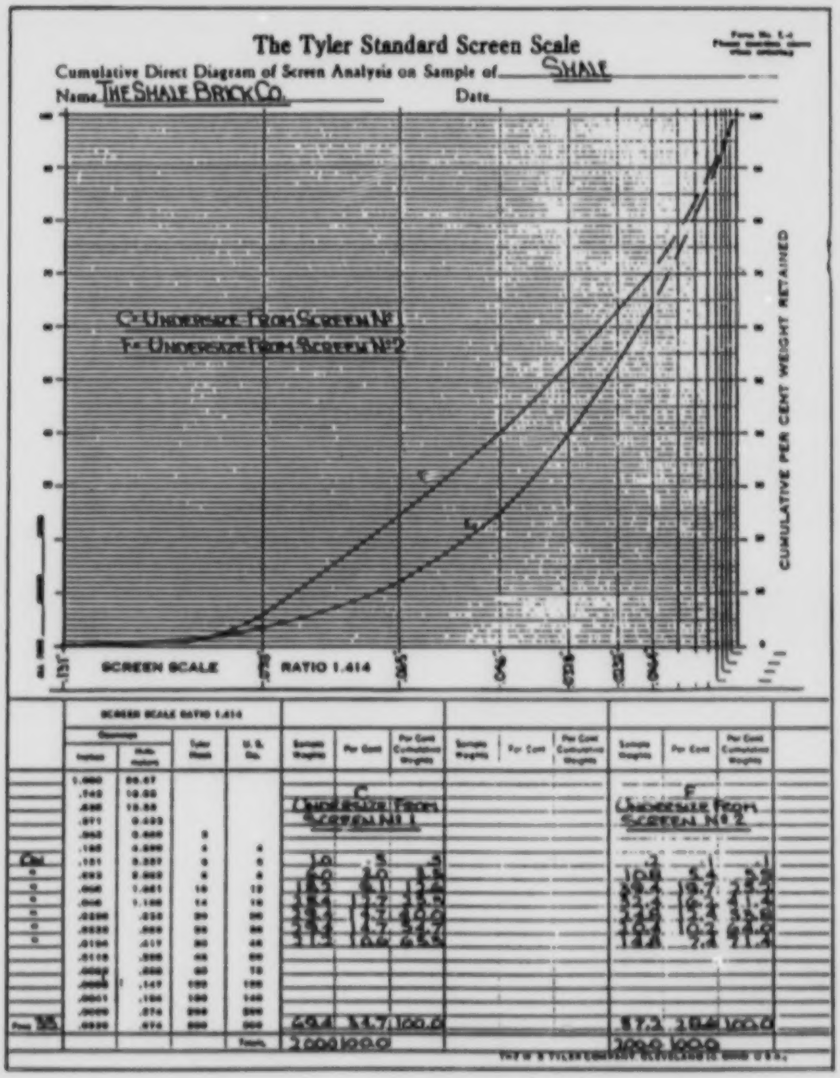
Form No. L-3

By cumulative per cent is meant the total amount of the product which would remain on a testing sieve if only one sieve were used for testing the whole sample, so that to get the cumulative weight, it is necessary to add all of the material which remains on sieves coarser than the one in question to the amount remaining on that sieve. In other words, on a cumulative plot, each point represents the total material that would be retained if only the one sieve represented by that particular point were used in the analysis.

HOW TO RECORD AND COMPARE SIEVE ANALYSES (cont.)

on the corresponding vertical lines drawn from the several openings on the horizontal scale of the diagram. After drawing in the curve, the percentages remaining on any set of openings other than those of the testing sieves used, can be found by interpolation (See Page 15) and in this way the re-distribution of the same material by any assumed set of openings can be determined.

The difference between a direct and logarithmic diagram is that in a direct plot the sieve openings are laid out to an arithmetical scale, while in the logarithmic plot they are laid down to a logarithmic scale. The peculiarity of the two scales is that in the arithmetical scale the actual horizontal distances between each vertical line increase or diminish in the same ratio as that in which the screen openings do, and in the logarithmic scale, the screen openings are plotted to the logarithm of the diameter of the opening (the ratio between the sizes being a constant), the horizontal scale of screen openings will be one of equal spaces.



The Tyler Standard Screen Scale

Cumulative Direct Diagram of Screen Analysis on Sample of SHALE

Name THE SHALE BRICK CO. Date

C= UNDERSIZE FROM SCREEN No 1

F= UNDERSIZE FROM SCREEN No 2

Form No. L-4, Cumulative direct plot.

HOW TO RECORD AND COMPARE SIEVE ANALYSES (cont.)

Cumulative direct plot

This form of direct-plotting is unique, all crushings being expressed on diagrams of same uniform dimensions and having the same length of base line. This renders a comparison of the curves from various crushings more comprehensive than when diagrams with varying lengths of base line are used.

The vertical lines on the diagram represent any given set of sieve openings having a constant ratio of 1.414, the first left-hand vertical representing the screen opening through which the sample has all been crushed, and the last, or extreme right-hand vertical, representing zero. The curves, therefore, start at the lower left-hand corner and terminate at the upper right-hand one.

By way of illustration, on the preceding page are shown in plotted form the sieve analyses of the undersize or products of the two screens in test No. 13-316 (See Page 10).

Both of these samples pass an opening of .131" (6 mesh). The vertical line at the left of the diagram was selected to represent that opening. The tabulations below the diagram give the data from which these curves have been developed.

Seven screens were used in these analyses, which are represented by the use of the same number of vertical lines on the diagram. The intersection of the curves with the vertical representing the opening. .0164" (35 mesh), shows that 65% of sample "C" is retained on a 35 mesh sieve with 35% passing, and that approximately 71% of sample "F" is retained on the same sieve. Since approximately 71% is retained and approximately 29% passes, the relative positions of the curves of sample "F" and "C" show that Screen No. 2 made the sharper separation, passing more of the particles that approached in size the opening of the screening surface employed in the commercial machine.

Had the separation been made at .065" opening (10 mesh) and the analysis carried through to .0164" opening (35 mesh), then the left-hand vertical would have been chosen to represent the opening .065" (10 mesh), and the fifth vertical to the right would then represent the opening .0164" (35 mesh).

After some experience with this method, the curve corresponding to any given material, size or method of crushing can be very closely approximated, and from it an estimate of the different sized quantities arrived at without recourse to any laboratory experiment on the ore.

Cumulative logarithmic plot

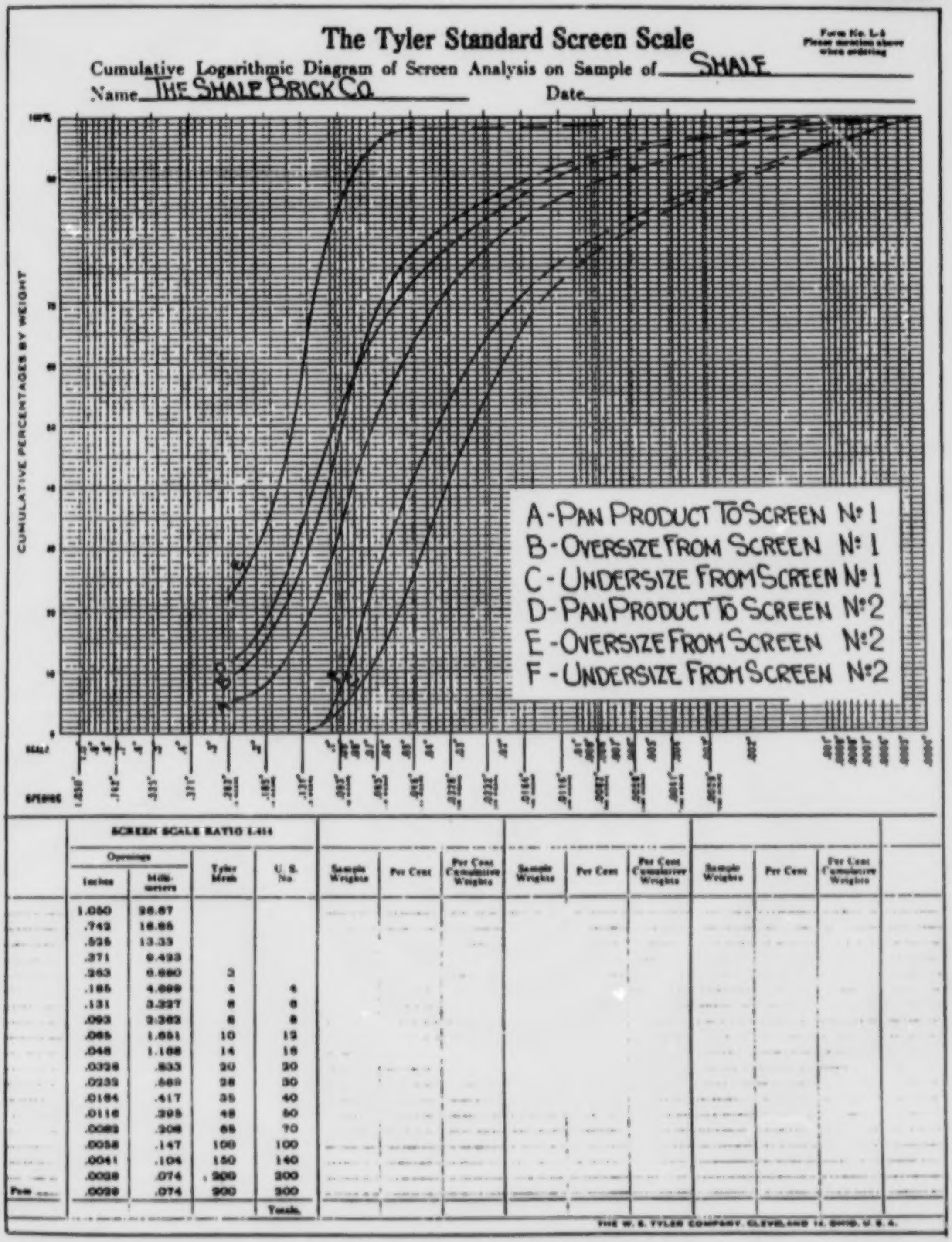
The arithmetical difference between the size of openings in the standard screens is small for the fine sizes, while for the coarser screens, the arithmetical difference is comparatively great. However, the ratios between the size of openings in successive sieves are constant and it is the ratios that are usually considered in commercial screening. A very ingenious and satisfactory way to plot a curve, covering a wide range of sizes, is by the use of a method in which equal distances on the horizontal scale represent equal ratios of diameter, thereby compressing the curve for the coarse sizes and extending it for the small sizes. This result is secured automatically by plotting the logarithms of the diameters of the openings on the horizontal scale. See illustration on the opposite page.

For those who are not familiar with logarithms, the only explanation that need be given to make this method of plotting useful and perfectly clear, is to say that although the logarithmic spacing is not the same as the arithmetical difference between the numbers, yet for short intervals the variation between the two is too small to be noticed, for example, .120-inch is located with practical accuracy on the paper by taking a point .8 of the distance from .1 to .125 (0.8 x 0.25—0.20), but for a longer interval, as from .1 to .2, the difference between the logarithmic and arithmetical methods is so great that it would not do to locate .150 half way between .1 and .2.

It will be observed that the logarithmic intervals are the same between 0.001 and 0.01 as between 0.01 and 0.1 or between 0.1 and 1.0.

The unit lines are heavier than the fractional lines. Where the space between units is small, as between .04 and .05 or .05 and .06, there is room for only one fractional line, but where the space is greater, as between .01 and .02 or .02 and .03, three fractional lines are shown, dividing the space into quarters, as .0125, .0150, .0175, .020 or .0225, .0250, .0275, .030. The lightest vertical lines indicate the correct position of the sieves of the Tyler Standard Screen Scale. It is a very easy matter to point the cumulative per cent of product on the light lines which indicate the sieves of the Tyler Standard Screen Scale and then draw in the curve. Curves for a series of analyses have been plotted by way of illustration on the diagram on the opposite page, the screen analysis being shown in tabulated form on Page 10.

HOW TO RECORD AND COMPARE SIEVE ANALYSES (cont.)



The Tyler Standard Screen Scale

Form No. L-5
Please mention above when ordering

Cumulative Logarithmic Diagram of Screen Analysis on Sample of SHALE
Name THE SHALE BRICK CO. Date

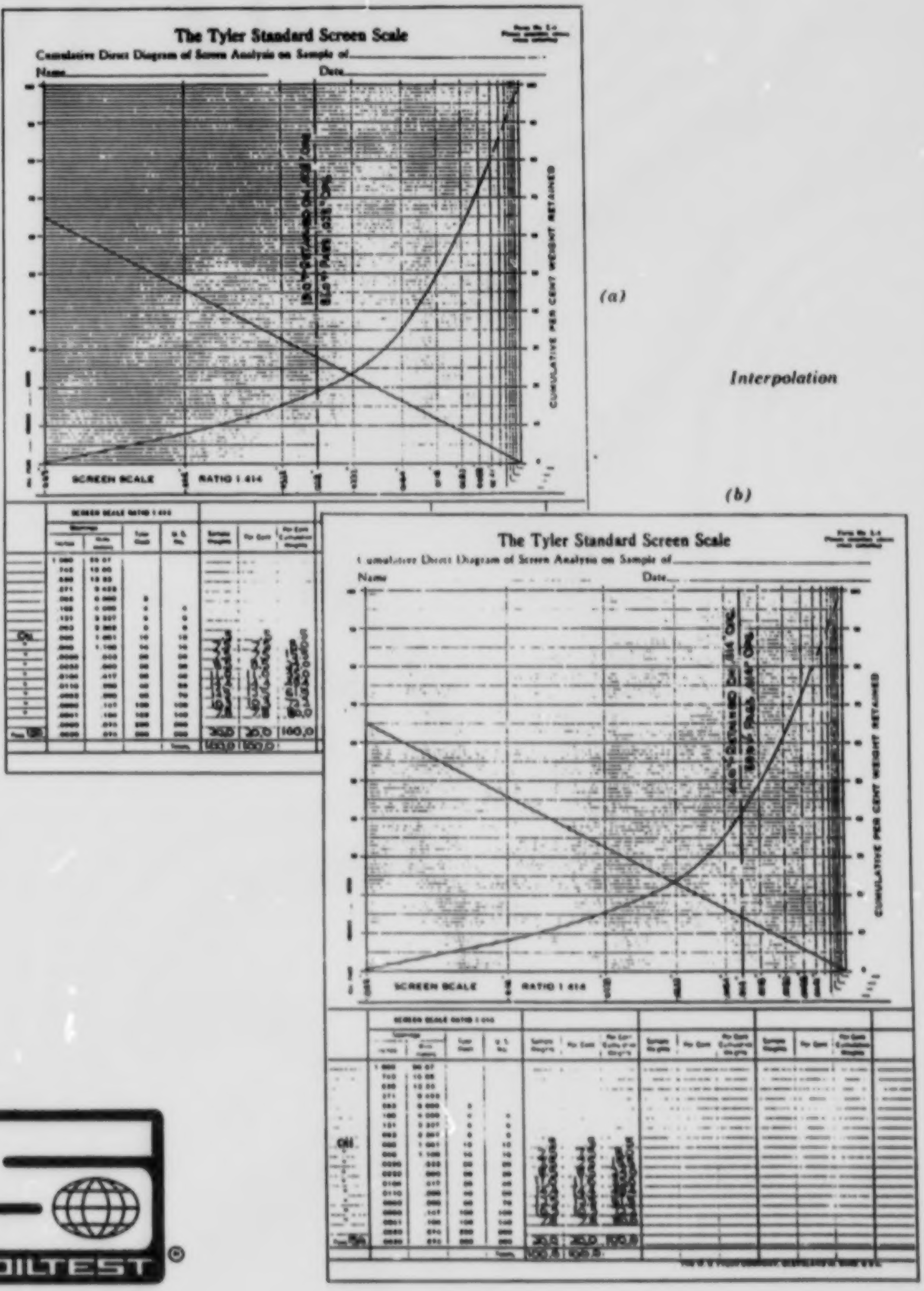
A - PAN PRODUCT TO SCREEN No. 1
B - OVERSIZE FROM SCREEN No. 1
C - UNDERSIZE FROM SCREEN No. 1
D - PAN PRODUCT TO SCREEN No. 2
E - OVERSIZE FROM SCREEN No. 2
F - UNDERSIZE FROM SCREEN No. 2

	SCREEN SCALE RATIO 1.414													
	Openings		Tyler Mesh	U. S. No.										
	Inches	Millimeters			Sample Weights	Per Cent	Per Cent Cumulative Weights	Sample Weights	Per Cent	Per Cent Cumulative Weights	Sample Weights	Per Cent	Per Cent Cumulative Weights	
	1.050	26.67												
	.742	18.85												
	.525	13.33												
	.371	9.423												
	.263	6.680	3											
	.185	4.699	4	4										
	.131	3.327	6	6										
	.093	2.362	8	8										
	.065	1.651	10	12										
	.046	1.168	14	16										
	.0328	.833	20	20										
	.0232	.589	28	30										
	.0164	.417	35	40										
	.0116	.295	48	50										
	.0082	.208	65	70										
	.0058	.147	100	100										
	.0041	.104	150	140										
	.0029	.074	200	200										
Pan	.0029	.074	200	200										
				Totals										

THE W. S. TYLER COMPANY, CLEVELAND 14, OHIO, U. S. A.

Form No. L-5, Cumulative logarithmic plot.
(Also supplied with MM Rulings—Form No. L-6)

HOW TO RECORD AND COMPARE SIEVE ANALYSES (cont.)



(a)

(b)

Interpolation

SOILTEST®

HOW TO RECORD AND COMPARE SIEVE ANALYSES (cont.)

Interpolation

Locating other sieve openings than those used in the test

It is easy with a cumulative direct plot to find percentages that will pass or be retained by openings other than those used in the sieve test and, reversely, to find what opening is required to pass, or retain, a designated percentage.

After the sieve test has been made, the vertical lines on the plotting paper should be marked at the base line with the openings of the sieves that were used, and the curve drawn in. (The vertical at the extreme right represents the pan.)

Then, on any vertical line (all of which are divided into 100 equal parts by the percentage scale) mark off the number of spaces indicated by the significant figures shown in the sieve opening at its base, as for example, if the vertical line is marked .065", mark off the point representing 65 divisions on the percentage scale. Then draw a diagonal line from this point to the pan or zero point on the base line.

In the illustration, the vertical at the extreme left has been used. As this vertical is marked .065" at the base line, 65 parts have been marked off on the percentage scale. Had the vertical .046" been used, 46 parts should have been marked off on the percentage scale of said vertical and the same diagonal located.

If the diagonal is drawn correctly, the number of parts set off at its intersection with any vertical will agree with the value at the base line.

In the illustration on the opposite page two examples are shown:

(a) To find what percentage will be retained by, or pass, an opening of .028 inches. Locate the horizontal line representing 28 in the percentage scale. At the place where this line intersects the diagonal, draw a vertical line perpendicular to the base. The point at which this vertical line crosses the sieve test curve shows that 19 per cent is coarser and 81 per cent finer than the opening .028 inches.

(b) To find what opening will retain 41 per cent and pass 59 per cent. Through the point representing 41 per cent on the sieve test curve, draw a vertical line perpendicular to the base. The intersection of this vertical line with the diagonal line shows the opening to be .014 inches.

Other Types of Plotting Paper

The plotting paper hereto described, all have the left and right hand ordinates in terms of percent retained on a direct scale with the abscissa scale going from right to left in ascending order of magnitude, ruled either direct or logarithmic.

Many laboratories studying gradation have their own method of plotting which they consider especially suited for the particular material being tested and for the specific purpose of the plot. Some of these methods are as follows:

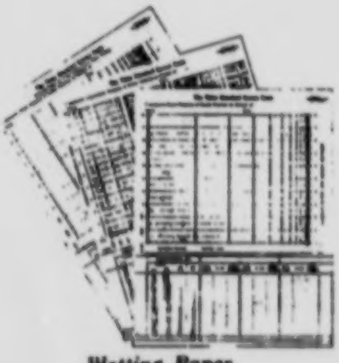
The National Crushed Stone Association prefers to use a plot with the left hand ordinate in terms of percent passing with the abscissa plotted to the scale of the square root of two and the sieve opening going from left to right in size of opening.

This is advantageous since gradation curves can be studied for maximum size aggregate as coarse as four inches.

Another popular type is to express the percent retained on the ordinate on a logarithmic scale with the sieve openings along the abscissa also on a logarithmic basis going from left to right in ascending order of grain size.

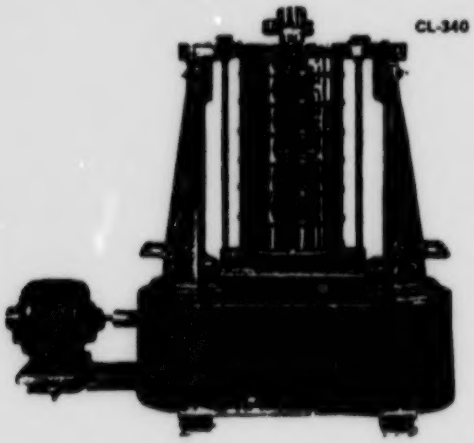
For particular industries special plots are required. As for example, in pulverized coal laboratories the Rosin and Rammler Plot has been found to be the most useful. This plot is particularly helpful in showing inadequate screening of the finer meshes and will quickly spot an error in the sieve test in this range.

If the regular Tyler Plotting Paper does not adapt itself to the users own method of plotting, please advise and our laboratory may be of assistance in suggesting other methods of obtaining the correct plot.

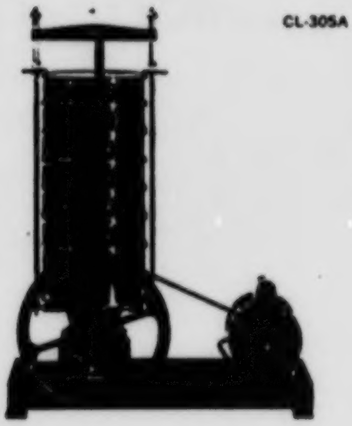


Plotting Paper

CL-300A



CL-340



CL-305A

MOTORIZED SIEVE SHAKER

The Motorized Sieve Shaker is an adaptation of the new Soiltest Hand-operated Sieve Shaker (Model CL-300A). Basic difference between the two is in the power requirement.

Ideal for projects demanding a large volume of sieve analysis such as is carried on in many types of commercial, university, engineering, research and highway department laboratories.

The Motorized Sieve Shaker utilizes horizontal and vertical motion along with tapping action—the same actions as those utilized by the hand-operated shaker. Spring-loaded tie-rods make the vertical motion possible. The shaker imparts a tap to the sieves at each change in direction as the sieve holders sway from side to side; in addition, during each tap an upward jolt takes place. Proper separation of all materials is possible with the tapping action, and the materials rush quickly through the sieve meshes. Sample testing time is greatly reduced. The combination of actions assures an accurate sieving or grading test on large quantities of material.

Sturdily constructed parts mounted securely on a base. Parts are plated to resist rust and withstand wear. Unit has a handsome metallic finish and is easy to clean and maintain.

Capacity of 7 U. S. Standard 8-inch diameter sieves with pan and cover. Base is 23 inches x 13⅛ inches; height is 33 inches. Available for different power requirements:

Models Available:

CL-305A-1	110-220V, 60 cycle, single phase AC.
CL-305A-8	110-220V, 50 cycle, single phase AC.

Shipping Weight: 75 pounds. *Net Weight: 60 pounds.*

RO-TAP SIEVE SHAKER

The Ro-Tap Sieve Shaker reproduces the circular and tapping motion given testing sieves in hand sieving. The uniform, mechanical action assures accurate and dependable tests.

Using the regular height 8 inch diameter sieves, one sample can be tested on a series of six sieves of different openings, all with one operation of the machine. By using pans with extended rims, three different samples can be tested at one time with regular height sieves.

The Ro-Tap is simply and durably constructed and is run by a ¼ horsepower electric motor. The running parts operate in oil. The machine is easily installed; no special foundation is required.

The dimensions of the Ro-Tap are: height, 32 inches; width, 34 inches; and depth, 17 inches.

CL-340	110 volt, 60 cycle, single phase AC
CL-340B	110 volt, 50 cycle, single phase AC
CL-342	220 volt, 60 cycle, single phase AC
CL-342B	220 volt, 50 cycle, single phase AC

Shipping weight: 450 pounds.

CL-300A HAND-OPERATED SIEVE SHAKER

The new streamlined SOILTEST CL-300A Hand-operated Sieve Shaker incorporates both horizontal and vertical motion with tapping action; spring-loaded tie-rods make vertical motion possible. The shaker imparts a tap to the sieves at each change in direction as sieve holders sway from side to side. Also, in each tap there is a slight upward jolt. The tapping action assures proper separation of all materials and a fast flow of material through sieve meshes. Shaking time for sand samples is greatly reduced. Presence of the horizontal and vertical motion together with the tapping action eliminates the need for any final hand shaking action frequently required when using other shaking equipment.

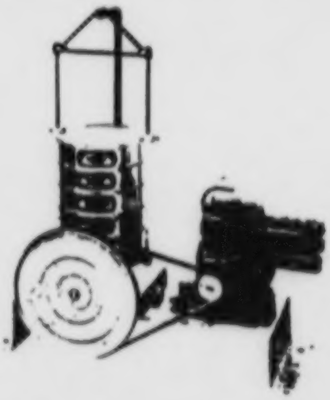
The sieve shaker is especially useful in the field where lack of power makes other units impractical. A specially designed drive system operating on a high pulley ratio assures easy hand action. The sturdy shaker can be mounted on a laboratory bench or table; there is ample clearance for mounting and operating on any flat surface. Fast-sliding holders adjust to tighten and hold the sieves in place during the shaking operation.

Unit is entirely self contained. Few working parts for easy maintenance. Parts are plated to resist rust. Handsome metallic finish.

The new Soiltest Model CL-300A Hand-operated Sieve Shaker has a capacity of 7 U.S. Standard 8 inch diameter sieves with pan and cover. Space requirements are 14 inches wide by 18½ inches long for the base. Height of the shaker is 30⅞ inches.

Shipping weight: 54 pounds. *Net weight: 42 pounds.*

CL-308 GASOLINE ENGINE FIELD SIEVE SHAKER



The Portable Field Sieve Shaker is driven by a small 4 cycle gasoline engine. All parts including the air cooled gasoline engine are mounted on a sturdy base plate. The shaker is lightweight and is designed specifically for field operations where electric power is not readily available. Model CL-308 has a capacity of 6 full height or 13 one half height sieves with the pan and cover.

A combination rocking and tapping action is imparted to the sieves through the drive mechanism. The few working parts of this portable sieve shaker make it easy to clean and maintain.

Shipping weight: 150 pounds.

TYLER TIMER

To obtain comparable results between two or more tests, each should be run the same length of time, and in order to terminate the tests at the time prescribed, an automatic timing device is necessary.

The *Tyler Timer*, shown in the accompanying illustration, stops the machine automatically after the time prescribed for the test has elapsed. Two types of Timers are supplied, one for the Ro-Tap Testing Sieve Shaker and one for the Ty-Lab Tester.

CL-345 Tyler Timer

TY-LAB TESTER

The *Ty-Lab Tester* is designed for testing large samples of materials as coarse as 4-inch. Soiltest Model CL-380.

On coarse sizes of rock, a test can be made in one or two minutes' time, the separations being practically as accurate as if each piece were tested in the openings by hand.

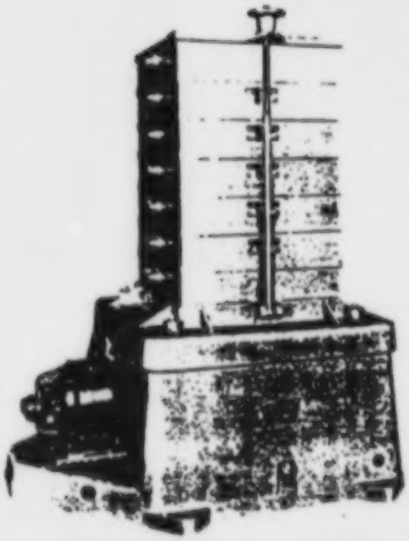
The Ty-Lab Tester handles as many as six 18" square sieves. When large samples are tested, the sixth sieve may be replaced by a collar which makes it possible to start with a double size sample.

After the sieves are placed in position, the rods which hold them in place are swung into position and the hand wheel is tightened to hold them firmly in place while the machine is operating.

Every move is automatically controlled and is exactly duplicated in every test so that the possibility of error due to the human element is entirely eliminated.

The Ty-Lab Tester is supplied with a ¾ hp motor of the required voltage and frequency.

The over-all dimensions are 2'9½" x 3'4½" x 4'3".



CL-380 Ty-Lab Tester with six 18" Square Sieves and pan

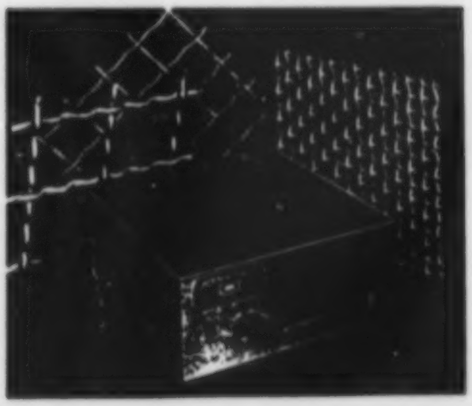
SQUARE FRAME PORTABLE FIELD TESTING SIEVES

The *Tyler Square Frame Portable Field Testing Sieves* are designed with interchangeable sieve bottoms. Sieve bottoms of any standard openings are available, but separations finer than 20 mesh are not recommended on this unit. Soiltest Model CL-320.

The frame equipped with the rocker bottom pan is recommended to catch the fines; however, removable rockers are also available.

The screens are held in place by quick acting clamps and can be changed quickly from one specification to another as required. All of the screens can be clamped into the frame for carrying or when not in use.

The sieve bottoms are approximately 12" square and the overall dimensions are 15¼" x 12¼" x 7¾" including the rockers.



CL-320 Portable Sieve ready for use

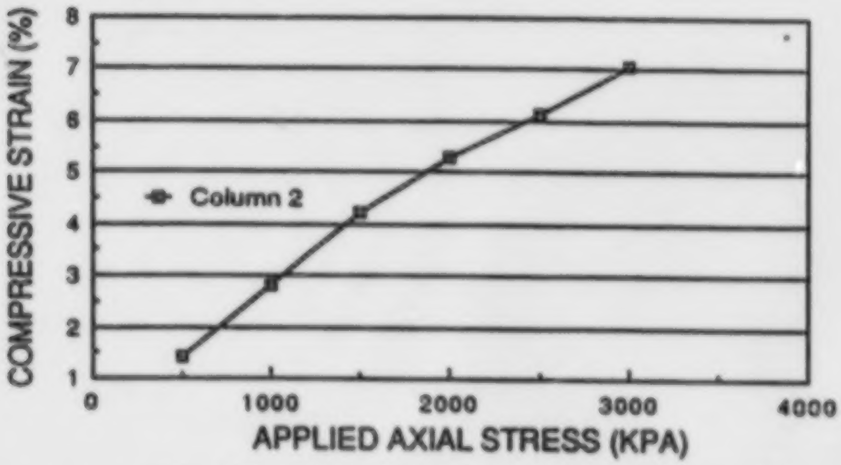
TEST: Laboratory Compressibility Analysis

EQUIPMENT NEEDS:

- Compression test frame capable of retaining 8 cm diameter by 4 cm high samples (as used for unconfined compression tests)
- Porous, brass filter base plate to facilitate sample drainage during loading and cylindrical sample holder
- Pneumatic or hydraulic pressure supply with pressure gauge or force transducer capable of registering peak axial loading stress to 10^4 kPa
- Axially-oriented displacement gauge
- Estimated equipment costs - $1000 (with load frame available)

TEST METHOD:

1) Test cylinders are lightly lubricated with non-reactive grease and placed on top of porous brass filter sections
2) Test samples are formed within the cylindrical holding tubes in a manner similar to that followed in preparing percolation rate sample materials; slurry materials are poured in place and left, undisturbed, to drain.
3) Uncemented samples are tested at specific bulk density conditions, typically those which result from placement and following percolation drainage
4) Sample heights are determined and the loading ram is placed in contact with the sample surface
5) Loading forces are applied in increments of 20 kN or as sufficient to maintain axial rates of deformation approximating 1 - 2% of the sample axial thickness per second
6) At each point of incremental loading, axial load, axial deformation and load frame deformation parameters are recorded
7) Following testing, compacted samples are retrieved, weighed moist, dried and then re-weighed to determine final moisture content conditions following stress loading and drainage
8) Test data is plotted in terms of compressive strain versus applied axial stress for drained samples for determination of compressibility moduli (kPa/% strain) parameters (a measure of resistance to wall convergence or back subsidence); typical test results are illustrated:



TYPICAL COMPRESSIBILITY TEST RESULTS

TEST: Laboratory Determination of Moisture Content and Liquid Limit Conditions

EQUIPMENT NEEDS:
- Casagrande liquid limit apparatus
- Weighing balance having weighing limit at 100 g ± 0.01 g

GENERAL:

Liquefaction of emplaced backfill is the catastrophic result of excessive moisture being retained within the fill, commonly resulting where the rate of filling exceeds the rate at which water can percolate through the fill mass. This problem is generally associated with uncemented tailings or natural sands but is not exclusive to them alone. Liquefaction occurs at the transition of fine or silty aggregate materials from a solid to liquid state.

"Quick" liquefaction occurs when an upward flow gradient of water exists such that the seepage pressure exerted within the fill exceeds the submerged weight of fill solids, causing the solids to lift. Where adequate percolation of water can be established within placed fill, a negative hydraulic gradient is established. When poor percolation is achieved, the vertical head of saturated fill can create a sufficiently high hydraulic gradient that reverse percolation (toward the free surface of the fill) will result.

"Cyclic" liquefaction occurs when saturated or near saturated fill material is subjected to dynamic stress loading. Under the action of vibration or shock, fill particles will settle and result in increasing pore water pressure effects as the fill voids become occupied by solids displacing contained liquid. If natural percolation cannot remove sufficient quantities of water per unit time, the induced pore pressure may exceed the submerged weight of fill solids, resulting in a loss of material shear strength and liquefaction. Such conditions occur coincident with seismic or blast-induced shock loading effects.

Soils which are susceptible to liquefaction exhibit typical size distribution characteristics, normally poorly graded, which are similar to those of tailings products. Similarly, where poorly graded tailings, comprising a high proportion of fines and which develop percolation rates below the generally accepted industry rate of 10 cm/hr, the potential for retention of water and liquefaction is high.

TEST METHOD:

Casagrande (Liquid Limit) Test

1) Equipment for this test consists of a cup, used to hold moist tailings or soil products, and a hand crank, used to raise and lower the cup/material through a measured height prior to impact on a base plate
2) A measured quantity of -40 μm tailings material is mixed at a fixed moisture content and is smoothed into the holding cup.
3) A groove, of specified profile, is cut into the material surface and the cup is raised and lowered until a minimum 1 cm length of the original groove is shown to close up.
4) The "liquid limit" is represented as the sample moisture content at which groove closure occurs after exactly 25 drops have occurred.
5) If the measured "liquid limit" moisture content approximates the in-situ fill moisture content, within 10%, then the liquefaction potential for the tailings material is assumed to be high.

The use of such tests is qualitative and is strongly time dependent. For example, typical tests should be performed on materials which have been permitted to undergo percolation for periods of at least 24 hours after initial placement, typical of minimum wait times usually set before stope access is permitted by work crews in cut-and-fill mining operations.

Moisture content analyses are performed by weighing total material weights both prior to and following oven drying. The moisture content condition, as is standard in the mining industry, is reported in terms of a percentage of the total (dry solids + moisture) sample weight.

TEST: Physical Modelling of Failure Mechanisms

EQUIPMENT NEEDS:
- Bench scale box models
- Large scale underground stope models
- Centrifuge assembly/stope block model frame

GENERAL:

Physical models, used to simulate scaled stope geometries and the mechanisms of anticipated backfill failure, are not commonly applied tools capable of being utilized by mine staff. Such tools, used primarily by researchers, comprise three general types: bench scale models (at 1:50 or 1:100 scale), large scale surface or underground models (at 1:10 or 1:50 scale) and centrifuge models (typically close to the bench scale model range of sizes).

Physical modelling, though not generally applicable by mine staff, is useful for examination of various failure phenomena, characteristic of backfill structures, and which may be studied to improve the placement quality of such fills.

Bench Scale Models

Generally having the form of rugged, timber boxes at dimensions approximating (0.6 m x 1.3 m x 1.3 m), such scaled models have been used to evaluate cement segregation effects, induced bulkhead pressure conditions and predicted stable heights of exposed fill faces which could be inferred to exist in-situ. Problems associated with such models primarily concern difficulty in ensuring that the effects being modelled are true physical representations of in-situ conditions rather than model size conditions. For scaling, therefore, limitations must be addressed, and typically involve factors including:
- determination of an accurate laboratory or in-situ material data base concerning backfill strength capabilities which will be expected to develop
- backfill materials must be down-sized such that the scale factor of the model geometry is accounted for while ensuring that excessive use of "clay" sized material is avoided
- proportional reduction in cement cohesion bond strengths must be incorporated to maintain the proper ratio of scaled height versus material strength that occurs in-situ; such reductions can be created by either reducing the cement weight proportion used in scale model tests or by reducing the curing interval at which models are tested

Large Scale Models

Large scale test models generally imply the use of models at scale factors less than 20-to-1. The advantage offered by such tests, however, is that truly representative backill materials, produced by the actual mine fill plant, may be used and placed for analysis. Also, due to the increased size of such models, a majority of scaling problems normally associated with smaller scale modelling tests can be avoided.

At such large model size requirements, the costs to perform modelling analysis of fill quality parameters becomes prohibitive (estimated to be in excess of $50,000 per test). Similarly, due both to a lack of significant historical experience in development of such models and to limitations in availability of trained research staff to conduct such tests, limited application of large scale model tests as a means of evaluating fill quality control parameters is anticipated.

Centrifuge Models

Centrifuge techniques may be utilized to subject geometrically-scaled fill models to stress and equivalent deformation conditions experienced by large in-situ masses acted upon by gravity and closure stress components. Though generally manufactured at scale sizes approximating 100-to-1, the action of centrifuging can be used to exert stresses upon corresponding points in the modelled structure similar to those which would naturally occur within the full-sized fill block in-situ.

A model, accelerated in a centrifuge, can be made to experience similar physical behaviour (distortion, arching, cracking and shear failure) as the actual fill mass. In limited tests conducted to date, scaled models, at 100-to-1 size factoring, have been subjected to stresses equivalent to 100 gravities acceleration. Where modelled laboratory materials, having suitably scaled strength and size parameters, can be shown to resist failure under the action of such stresses, they would also exhibit suitable cementation strength to resist failure in-situ.

As with large scale model capabilities, limited centrifuge facilities are available in Canada with which quality assessment tests can be conducted. Similarly, due to the high expense and technical requirements necessary to build and maintain such a design oriented tool, the likelihood of mining groups being able to acquire such hardware is very low. Where used, however, such modelling is capable of analyzing variable stope geometry effects and the effects of wall closure on stope fill mass stability. Where scale modelling is to be considered for use in characterizing fill properties, the use of the centrifuge technique would appear to be the most viable of the several different techniques presently available.

TEST: In-Situ Physical Sampling Techniques for Backfill Specimen Recovery

EQUIPMENT NEEDS:

- For non-continuous sampling of in-situ materials, driving of thin-walled soil sampling tubes (Shelby type) is necessary.
- Where more cohesive fill is to be sampled, thick-wall tube samplers may be required, though more fill disturbance results.
- Casing materials will be required for cohesionless material sampling.
- Samplers and casing materials will require the use of impact devices (hammers), pressure jacks (hydraulic or pneumatic) capable of generating up to 150 kN force or electrohydraulic vibratory-drive drive installation tools
- Sample extraction jigs are necessary to provide release of specimens from sampling tubes
- For continuous sampling of in-situ materials, traditional hollow tube rotary drilling can be used; optimum recovery requires use of electric, hydraulic or pneumatic rotary drills with triple tube, wire line capability and casing protection.
- To avoid saturating fill samples at the drill face, compressed air may be recommended to clear cuttings, though sample drying results
- Polymer chemical addition to the flush water may help reduce core friction wear

SAMPLING PROBLMS:

- Sample extraction from casing or tubes is difficult due to material swell during the physical process of collection
- Where borehole drilling fluids are utilized to aid the drilling process, sample saturation conditions may be significantly altered, as are the significant physical properties which are to be measured
- In most low cement content fills, without casing protection, the drill hole can suffer collapse during the process of tube extraction and therefore cannot be re-used
- Unless multiple inner sampling tube systems are utilized, captured samples suffer from severe breakup during rotation and extraction
- Less than 100% material recovery takes place
- Generally disturbed and non-representative material is collected

TEST: In-Situ Deformation Modulus Analysis using Pressuremeters/Dilatometers (Qualitative Analysis)

EQUIPMENT NEEDS:

- Pressuremeter/dilatometer (at diameter sizes ranging from 22 to 74 mm) and pressure or volume flow monitoring unit
- For insertion into fill, one of:
  a) Pneumatic or hydraulic borehole drilling apparatus
  b) Pressure jack (hydraulic or pneumatic) capable of generating up to 150 kN force or electrohydraulic vibratory-drive casing installation tool
  c) Self-boring pressuremeter

TEST METHOD:

1) Pressuremeter-type devices are used to qualitatively determine shear and deformation modulus and shear strength data through comparison of field-derived data with laboratory-derived data.
2) Probes, prior to use, are calibrated within steel confining jackets.
3) Where the fill is very competent, boreholes are drilled to depth; where low fill strength exists, either self-boring types or vibratory penetration of hole casing lengths can be attempted. Holes may be drilled either horizontally or vertically, though ease of testing improves when vertical holes are used.
4) The probe pressure cells are lightly inflated to seat the probe casing.
5) Between 10-20 equal pressure increments are applied to the diametrally-expanding probe cells until the initial cell volume has approximately doubled.
6) Data (radially-acting pressure versus change in cell volume) is plotted and used to determine the in-situ Shear and Deformation (Young's) Modulus for the material.
7) Samples of typical fill material are also recommended to be manufactured under laboratory conditions and subjected to physical property testing in order to compare results; lack of data reliability exists due to known effects of fill disturbance caused by methods of pressuremeter installation presently used. Typical pressuremeter results for in-situ tests are represented below.

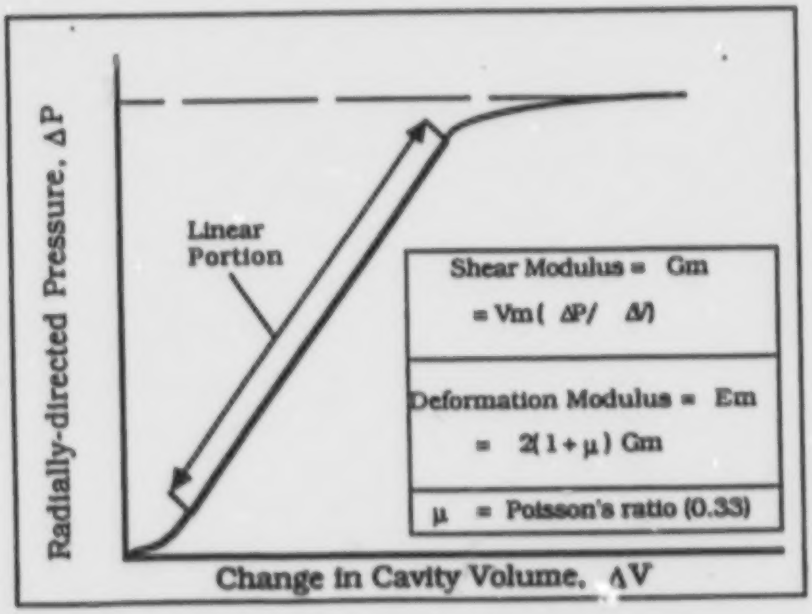


Plate Bearing Test (Quantitative Strength Analysis)

EDS:
- Shovels, pneumatic chisels etc. for site excavation of test material
- Loading jack (1) at 100 tonne capacity to provide normal loading force on backfill surface, minimum stroke @ 25 mm; jack capacity is estimated at 5 times the known material strength
- Universal loading column (tubular aluminum) for normal force application
- Steel bearing plate having minimum circular diameter = 700 mm (or set at 10 x the largest backfill aggegate particle diameter) and thickness = 25 mm
- Dial gauges or linear transducers (2) for measurement of average specimen axial deflections, anchored separate from plate installation
- Load cell @ 100 tonne capacity for axial load measurement
- System schematic illustrated in FIGURE I-1 (following)

A sessment

):

ite selection may either be on top of the fill mass, if access can be gained (as in ut-and-fill operational investigations) or within the fill block where more :presentative mass features would be exhibited. Fill surface sites are not enerally desirable as they are not representative of the gross fill mass condition. s with other in-situ tests, site selection should follow test drifting into the fill aass (by drill/blast, mechanical road header or manual cutting methods), referably along the long stope axis from the fill limits towards the central fill one. Test drifting permits staff to review changes in fill character within the tope and to select most representative test sites of average stope conditions.

At a selected site (to be cut aproximately 3.0 metres high), the back should be reinforced with support timber framing; the bearing floor requires removal of sufficient surface fill material to insure that the bearing surface is influenced solely by factors resulting from the fill pour (factors such as moisture content, material being undisturbed by excavation etc.).

When initially excavated, the floor must be covered by plastic sheeting to prevent drying effects during equipment setup.

Excavate a flat, circular test zone to a depth of 15 - 20 cm below the existing material floor surface having a diameter equal to twice the bearing plate diameter.

Prepare a flat roof bearing surface which will be approximately 3 - 4 times the bearing plate diameter in order that no failure will take place during testing.

A 20 cm grout layer is poured within formwork on top of the bearing surface , if the fill is unconsolidated or weakly consolidated, and the steel bearing plate is imbedded, level, into it. Curing is permitted to occur for 48 hours.

The load cell, bearing pads, loading jack and support column are installed perpendicular to the bearing plate surface; timbers are placed on top to act as cushions and as gap fillers.

Displacement transducers are installed at a distance from the loading assembly to minimize the effects of load column movement on measured fill displacements.

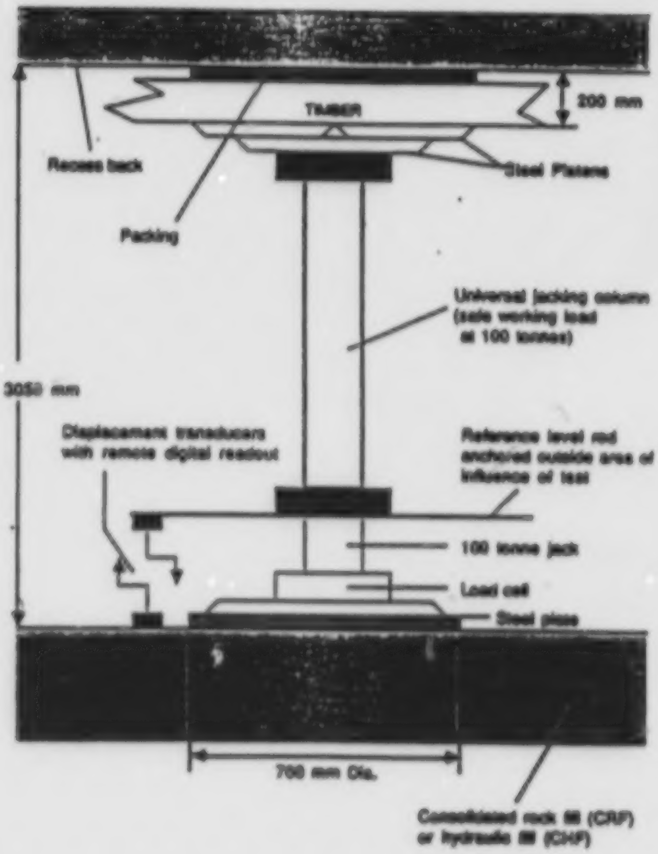
) Axial stresses are applied in small, 0.1 MPa increments for the first 1 MPa range to insure proper equipment seating; thereafter, stresses are increased in 0.2 MPa increments for an additional 1 MPa range until sample bearing surface failure is indicated. All stresses to be applied using fixed and linear strain rate application, approximating 2% strain per minute.

1) Following stress loading to 2 MPa total, stress increments of 0.3 MPa are carried out until loading jack deflection range is exceeded.

2) After each incremental stress application, axial displacement readings are taken when displacements have stabilized.

3) Tests are terminated by gradually reducing axial stress in 0.5 MPa increments, with subsequent displacement readings being taken at each step until complete stress relief occurs.

14) Recycling tests (rebound stress/strain response) may be performed by repeating the previous steps using 4 - 5 stress increments at 0.5 MPa each up to the point of material failure in each cycle.
15) The point at which the load/deformation or stress/strain curve slope alters provides an indication of the material bearing capacity (q_a): using various and different bearing plate sizes, techniques are also available by which the bulk cohesive strength (C) and internal friction angle (ϕ) for the fill material may be evaluated.



TIMBER
200 mm
Recess back
Steel Platens
Packing
Universal jacking column (safe working load at 100 tonnes)
3050 mm
Displacement transducers with remote digital readout
Reference level rod anchored outside area of influence of test
100 tonne jack
Load cell
Steel plate
700 mm Dia.
Consolidated rock fill (CRF) or hydraulic fill (CHF)

FIGURE I-1 - Test Configuration for In-Situ Plat
Bearing Test (after Gonano, 1977)

TEST: In-Situ Direct Shear Strength Analysis (Quantitative Strength Analysis)

EQUIPMENT NEEDS:

- Shovels, pneumatic chisels etc. for site excavation of test material
- Loading jacks (3) at 100 tonne capacity each; one to provide normal loading force, one to provide shear loading force
- Pin-jointed universal loading column for normal force application
- Metal shearing frame and inclined shear force bearing plate
- Two load cells, one each to measure shear and normal forces applied
- Four dial gauges or linear transducers for measurement of average specimen deflections axially and horizontally during shear failure
- System schematic illustrated in FIGURE H-2 (following)

TEST METHOD:

1) As with other in-situ tests, site selection follows test drifting into the fill mass , preferably along the long stope axis from the edge of the fill mass inward toward the central fill cone, either in order to locate typical fill mass site conditions or to permit sampling of variable site conditions.
2) An area offset from the test drift will be excavated which should be ≥ 9 times the cross-sectional area of the test sample to be tested; during excavation, the back and walls must be supported using timber sets.
3) In the middle of the offset area, a square sample is prepared at height/width ratio of 1/2 (generally 0.5 m high by 1.0 m wide) leaving a height between sample and back approximating 2.5 m to accommodate loading eqipment.
4) A metal shearing frame is placed around the square sample and a normal force bearing plate is installed on top of the sample, as illustrated.
5) Normal and shear loading rams (plus optional load cells) are located and wedged against the back and side walls of the excavation. Normal forces are applied using a pin-jointed loading column to minimize friction forces in the shear direction.
6) Displacement measuring devices are located at the sample mid-height and axially-centred to measure average sample shear and normal (axial) displacements in the directions of applied stress.
7) During tests, incremental loading of the shear force system occurs, under fixed normal loading force, until failure occurs. An estimated normal stress is applied which approximates one-third the sample unconfined compressive strength.
8) Shear stress increments approximating 0.02 MPa are applied until failure occurs. After each shear stress increment, displacement measurements (horizontal and vertical) and stress values (normal and shear) are recorded.
9) Following initial sample shear failure, residual shear response may be determined by subjecting this same sample to normal and incremental shear loading. Up to three such residual strength tests can be performed with confidence.
10) Stress data is plotted in a similar fashion to laboratory data: the results of shear versus normal stress plots are utilized to derive in-situ material cohesive strength (C), internal friction angle (ϕ) and unconfined compression strength parameters. Typical plot output is illustrated.

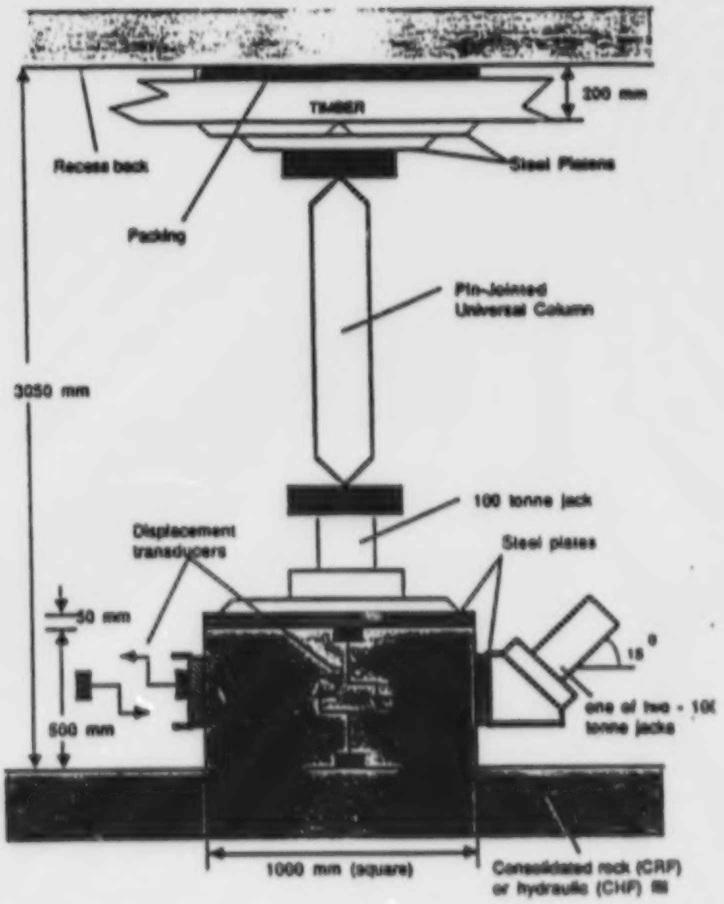
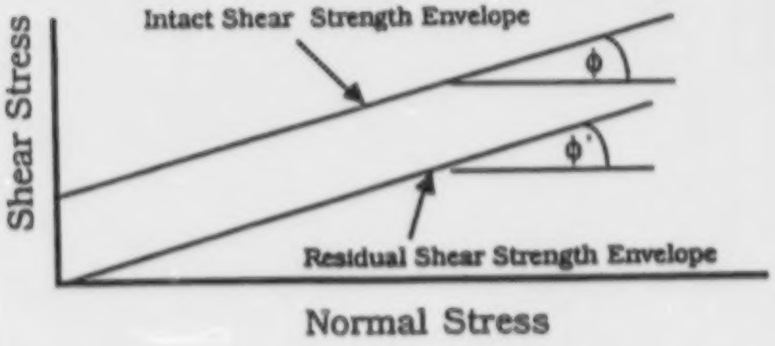

TIMBER
200 mm
Recess back
Steel Platens
Packing
Pin-Jointed
Universal Column
3050 mm
100 tonne jack
Displacement
transducers
Steel plates
50 mm
15°
one of two - 100
tonne jacks
500 mm
1000 mm (square)
Consolidated rock (CRF)
or hydraulic (CHF) fill

FIGURE H-2 - Test Configuration for In-Situ Direct Shear Test (after Gonano, 1977)



TEST: In-Situ Uniaxial Compression Test (Quantitative Strength Analysis)

EQUIPMENT NEEDS:

- Shovels, pneumatic chisels etc. for site excavation of test material
- Loading jacks (4), either air/hydraulic or electric/hydraulic, at 100 tonne capacity to provide normal loading force on backfill surface, minimum stroke @ 25 mm
- Steel bearing plate at base having average cross-section = 1000 mm (or set at 10 x the largest backfill aggegate particle diameter) and thickness = 10 mm
- Steel transfer platens (8), timber and grout for upper bearing surface contact loading
- Dial gauges or linear transducers (4) for measurement of average specimen axial and lateral deflections, anchored between the specimen ends and sides at mid-specimen locations
- Load cells (4) @ 100 tonne capacity each for axial load measurement
- System schematic illustrated in FIGURE J-2 (following)

TEST METHOD:

1) Site selection is preferred to be made within the fill block where representative mass features are exhibited. Fill surface sites are not generally desirable as they are not representative of the gross fill mass condition.
2) As with other in-situ tests, site selection should follow test drifting into the fill mass (by drill/blast, mechanical road header or manual cutting methods), preferably along the long stope axis from the fill limits towards the central fill cone. Test drifting permits staff to review changes in fill character within the stope and to select most representative test sites typical of average stope conditions.
3) Sites to be selected must be thought to be homogeneous for depths into sidewalls equal to the recess volume required. Sample sizes (approximately 4 - 6 orders of magnitude larger than unconfined specimens used for laboratory testing) show typical height-to-width ratios at 2-to-1 and dimensions in-situ approximating 1000 mm (width) by 2000 mm (height).
4) Maximum sample width will be generally selected on the basis of scaling of the largest aggregate particle diameter (8 - 10 times this size).
5) At a selected site, cut an opening into the drift side having dimensions roughly 9 times greater than the average sample area and approximately 3.0 metres high.
6) Within the excavation zone, and surrounded by support timber framing, the middle volume will be gradually carved into the shape of the final sample while always insuring that this material is kept wrapped in plastic to minimize moisture loss.
7) Prepare a flat roof bearing surface by carving the fill material surface, casting a grout reaction layer and installing timber/steel loading plates; the upper reaction surface area must be kept at least 1.5 times larger than the sample loading area to insure that failure of the recess back does not occur during testing.
8) A similar bottom reaction surface, consisting of grout layer and 10 mm thick steel plate, is installed on the sample surface to insure uniform stress distribution across the sample width during load application.
9) Two axial displacement transducers are used to average deflections between the level of the base loading plate (sample top) and the site floor (sample base); cantilever arms should be used to offset transducers away from the sample sides to prevent obstruction to other equipment or damage caused by failing material during test performance.

10) Lateral displacement may be measured in several ways, though the simplest method would require that fine holes be drilled horizontally through the mid-span of the sample, in two orthogonal directions. Fine potentiometers could be installed through such holes to measure lateral outward deflections which would result during sample loading.

11) Data which must be recorded prior to failure testing includes:
    - initial average height and width of sample
    - initial load cell value
    - initial axial and lateral deformation transducer readings

12) Loads are applied to the sample at uniform, linear rates approximating 2% of axial strain per minute; after each load increment is applied, readings are taken from each of the load and displacement transducers.

13) When peak axial loads have been achieved, loading stresses can be reduced incrementally while continuing to record data; cyclic re-loading tests can then be performed to evaluate the rebound loading capabilities of the material.

14) Bulk samples should then be recovered for determination of moisture content and bulk density parameters; in addition, data which is measured during this test can be used to derive the bulk unconfined compressive strength, peak strain at failure, Young's Modulus and Poisson's ratio parameters for the fill mass material.

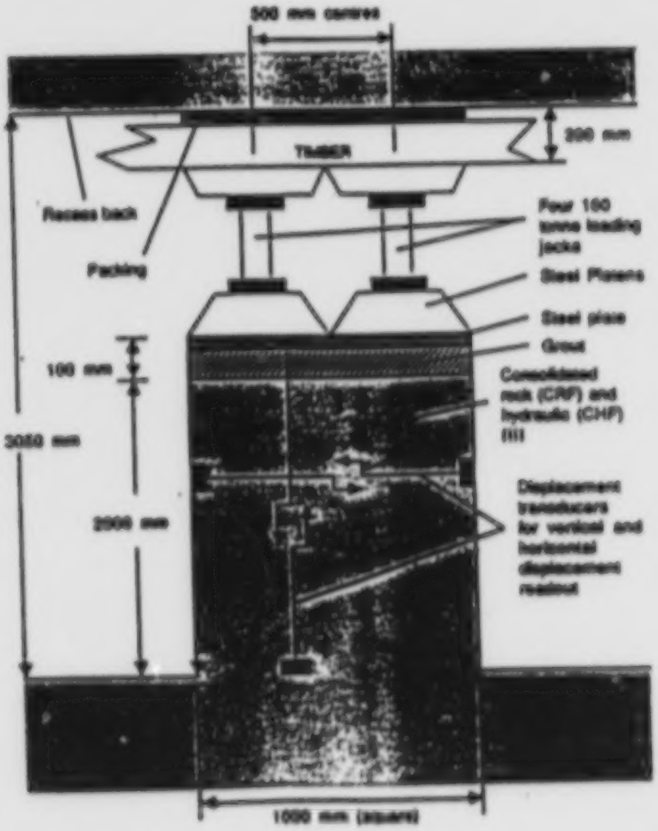

500 mm centres
200 mm
TIMBER
Recess back
Packing
Four 100 tonne loading jacks
Steel Platens
Steel plate
Grout
100 mm
Consolidated rock (CRF) and hydraulic (CHF) fill
3050 mm
2000 mm
Displacement transducers for vertical and horizontal displacement readout
1000 mm (square)

FIGURE J-2 - Test Configuration for In-Situ Uniaxial Compression Test (after Gonano, 1977)

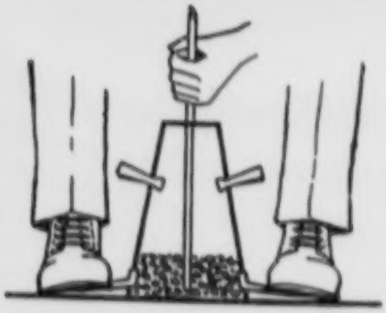
CONCRETE SLUMP TEST

Purpose of test: *To determine the consistency of fresh concrete and to check its uniformity from batch to batch. This test is based on ASTM C 143: Standard Method of Test for Slump of Portland Cement Concrete.*

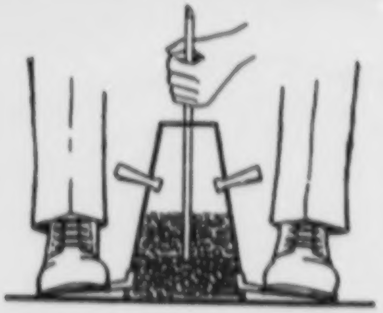
Take two or more representative samples—at regularly spaced intervals—from the middle of the mixer discharge; do not take samples from beginning or end of discharge. Obtain samples within 15 minutes or less. *Important:* Slump test must be made within 5 minutes after taking samples.

Combine samples in a wheelbarrow or appropriate container and remix before making test.

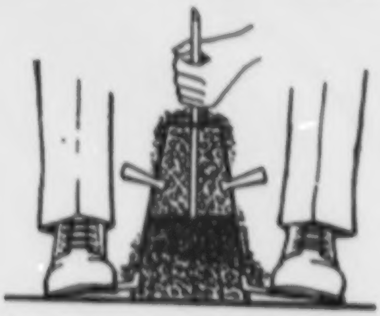
Dampen slump cone with water and place it on a flat, level, smooth, moist, non-absorbent, firm surface.



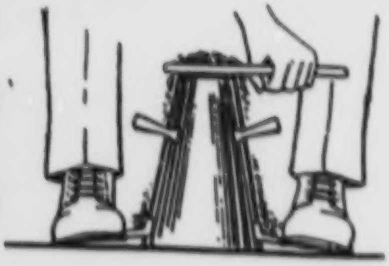
Stand on two foot pieces of cone to hold it firmly in place during Steps 1 through 4. Fill cone mold 1/3 full by volume [2-1/2" (63.5mm) high] with the concrete sample and rod it with 25 strokes using a round, bullet-nosed steel rod of 5/8" (16 mm) diameter x 24" (610 mm) long. Distribute rodding strokes evenly over entire cross section of the concrete by using approximately half the strokes near the perimeter (outer edge) and then progressing spirally toward the center.



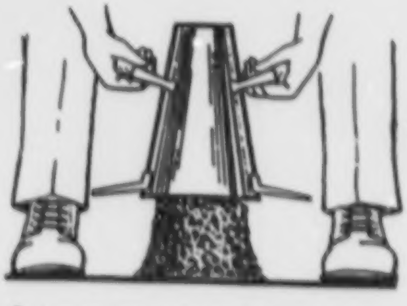
2. Fill cone 2/3 full by volume [6" (23 mm) or half the height] and again rod 25 times with rod just penetrating into, but not through, the first layer. Distribute strokes evenly as described in Step 1.



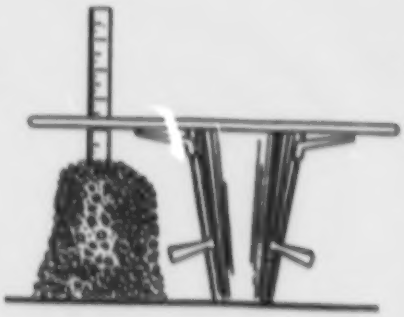
3. Fill cone to overflowing and again rod 25 times with rod just penetrating into but not through the second layer. Again distribute strokes evenly.



4. Strike off excess concrete from top of cone with the steel rod, so that the cone is exactly level full. Clean the overflow away from the base of the cone mold.



5. Immediately after completion of Step 4, the operation of raising the mold shall be performed in 5 to 10 sec. by a steady upward lift with no lateral or torsional motion being imparted to the concrete. The entire operation from the start of the filling through removal of the mold shall be carried out without interruption and shall be completed within an elapsed time of 2-1/2 minutes.



6. Place the steel rod horizontally across the inverted mold, so the rod extends over the slumped concrete. Immediately measure the distance from bottom of the steel rod to the original center of the top of the specimen. This distance, to the nearest 1/4 inch (6 mm), is the slump of the concrete.

APPENDIX I

INSTRUMENTS FOR IN SITU MEASUREMENT OF FILL PROPERTIES

SERATA GEOMECHANICS, INC.

SPDR TECHNOLOGY®

Serata Geomechanics, Inc.
4124 Lakeside Drive
Richmond, California 94806

15 January 1989

T-SP-89001

Preliminary data sheet

SERATA GEOMECHANICS, INC.

STRESSMETER S-200

The second-generation Stressmeter S-200 incorporates improvements derived from application of the first-generation Stressmeter (S-100) over the past five years. The S-200 probe is used to determine stress states in irregular, prefractured ground where conventional methods are not effective. The S-200 probe is applied within an NX test borehole, underground or from the surface, to measure stress states and material properties simultaneously in complex ground. The probe is equipped with an elastomer loading cylinder that can be safely pressurized up to 70 MPa (10,000 psi) in a borehole. Application of the probe is based on the Double Fracture Method (patent pending) of observing the failure and post-failure behaviors of fracture planes in relation to the pressure loading.

Four pairs of high-precision sensors (LVDTs) measure the borehole diametral deformations at 45° intervals in relation to the applied pressure. Magnetic and gravity sensors in the probe monitor the S-200 orientation in three dimensions. An important application of the S-200 is the planning and implementation of major earth construction work, including cost estimating, design analysis, quality control, and long-term safety assurance.

Applications

- Measurement of stress states and material properties in complex ground of brittle/fractured hard rock and ductile/yielding soft rock.

- Quantitative site characterization for construction and calibration of FEM computer model of complex earth structures.

Features

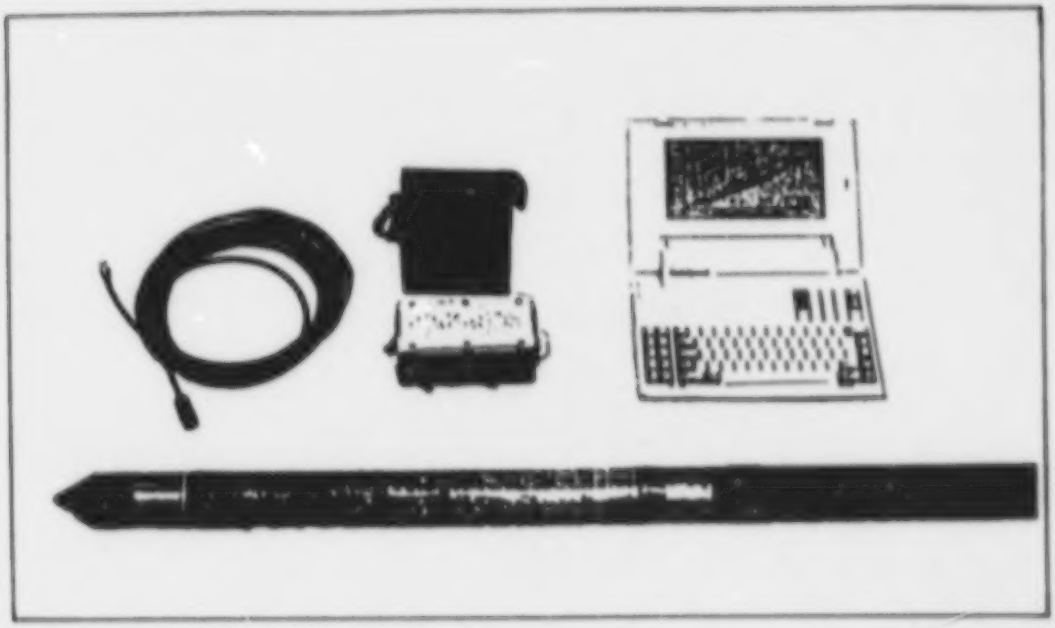
- Data processing capability built into the probe, enabling long-distance application with single conductor communication.

- Laptop computer operation of probe at the test site, providing instantaneous graphic analysis and display during the measurement process.

Specifications

Probe diameter: 73 mm
Probe length: 2.19 m
Weight: 22 kg
Borehole diameter: 76 ± 1.5 mm
Maximum oil pressure: 700 kgf/cm^2

Pressure resolution: 0.3 kgf/cm^2
Displacement resolution: 0.001 mm max
Orientation sensors range: 360°
Orientation resolution: ± 2°
Range of deformation measurement: 10 mm



Stressmeter S-200 with accessories

STRESSMETER S-100*

Description

The *Stressmeter S-100* is made to measure both stress states and material properties simultaneously in complex ground where no conventional methods are very effective. The probe is equipped with a self-contained soft elastomer loading cylinder* which can be safely pressurized up to 70 MPa in a borehole. The application of the probe is based on a new method* of observing failure and post-failure behaviors of the borehole with resolution as low as 1 micron.

A set of 4 diametral deformation sensors (LVDT's) embedded in the cylinder at 45° angular intervals measure the diametral deformations in four directions simultaneously, in relation to the applied pressure. The Field Computer (FC-100) is used to process and plot the data for stress analysis on site quickly and effectively. The probe and accessories are shown in Fig. 1.

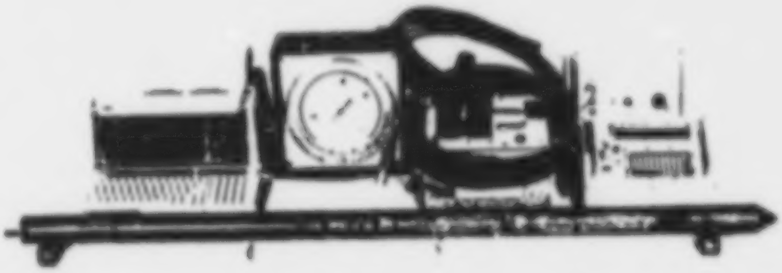


FIG. 1. Stressmeter (S-100) and Accessories

Applications

- Measurement of stress states using a single borehole without requiring overcoring or hydrofracturing.
- Stress measurement in brittle hard rocks containing pre-existing fractures.
- Stress measurement in ductile yielding media such as hard soil and soft rock.
- Acquisition of accurate inputs of stresses and properties for use in finite element analysis and design of complex earth structures.

Features

- Simple, direct and computerized method of stress measurement in complex ground without requiring the unrealistic assumptions of elasticity and uniformity.
- Measurement of in situ mass material properties of rocks, including strengths, elastic coefficients and fracture and creep characteristics as a by-product of the stress measurement.

---

*Both method and apparatus, patent pending internationally

Analysis

In brittle ground, loading is applied first to consolidate the pre-existing fractures and then to create a pair of mutually perpendicular radial fractures which freely separate at reloading pressures p_1^f and p_2^f. The stress state (P_0 and Q_0) is determined from the relation:

$$p_1^f = 3Q_0 - P_0 \qquad P_0 = \text{maximum principal stress and}$$

where:

$$p_2^f = 3P_0 - Q_0 \qquad Q_0 = \text{minimum principal stress}$$

In both brittle and ductile ground, the stress orientation is determined from the diametral deformation ellipse in which P_0 and Q_0 coincide with the maximum and minimum diametral deformation.

In ductile ground, the hydraulic loading causes diametral expansion due to yielding and creep deformation. The stress states are back-analyzed from the deformation and creep properties utilizing the Rheological Element Method computer program (REM) especially developed for this application.

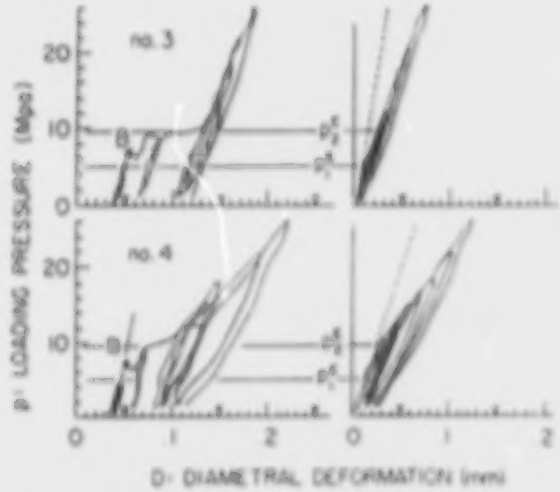


FIG. 2. S-100 Test Curves in Brittle Rock Showing Double Fracture Free Separation Pressures p_1^f & p_2^f.

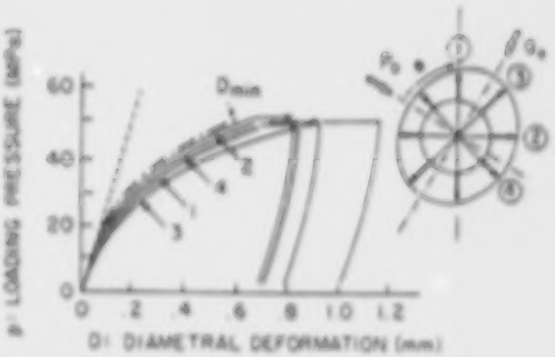


FIG. 3. S-100 Test Curves in Soft Rock Showing Yielding Expansion of Diameter Followed by Creep Deformation.

Specifications

Probe diameter:	99mm	Borehole Diameter:	104 ± 1.5mm
Probe length:	2.08m	Maximum oil pressure:	700 kgf/cm^2
Weight:	57kg	Displacement resolution:	.001mm max.

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PROPERTYMETER P-100*

Description

The *Propertymeter P-100* is a borehole penetrometer probe equipped with a set of 8 loading pistons which perform 8 simultaneous punch tests on the wall of a borehole. The 8 tests can be carried out in 10 to 15 minutes at a given position. With speed and efficiency the probe can determine in situ material properties and their distributions along the length of a borehole. It is especially effective in identifying anomalies and weaknesses in complex ground. The Field Computer (FC-100) is used for processing and quality control of the measurement.

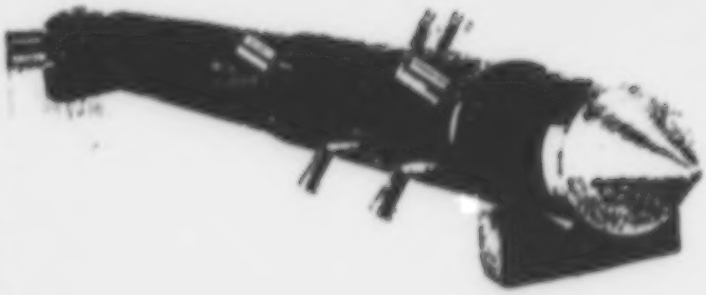

FIG. 1 Propertymeter (P-100) Probe

Applications

- Measurement of in situ rock properties in complex ground.
- Determination of material property profile through test holes identifying nature and scope of ground weaknesses.
- In situ experiments on ground weaknesses such as fractures, clay seams and other anomalies.

Features

- Obtains realistic in situ material properties critically needed for finite element simulation analysis.
- Provides good quality control for the measurement data with on-site data evaluation.
- Low cost and high speed measurements achieved by comprehensive computerization on site.

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*Patent Pending Internationally

Analysis

The data is plotted using the Field Computer, FC-100 in the form of piston penetration (P) vs. loading pressure (p). A set of 8 P-p curves are obtained from each loading test. The curves are used to determine deformation modulus, Young's modulus and failure strength as well as post-failure behavior as shown in Fig. 2. Post failure penetration of the pistons discloses fracture characteristics as well as triaxial properties of the ground. Whenever needed, creep properties are determined by conducting creep tests with the pistons as shown in Fig. 3. For an accurate analysis of the creep and post failure tests, the computer simulation method is applied.

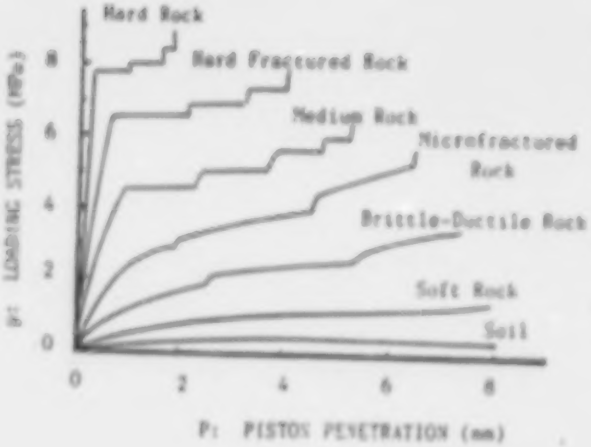


FIG. 2. Compendium of Piston Penetration Curves in Various Rock Media.

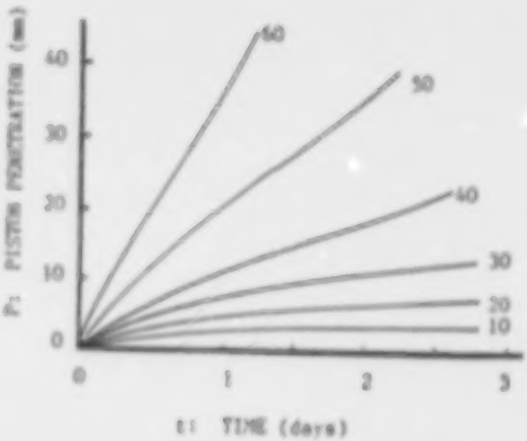


FIG. 3. Compendium of Penetration Creep Curves Illustrating Method for Determination of Creep Properties of Ground.

Specifications

Probe diameter:	99mm	Borehole I.D.:	104 ± 1.5mm
Probe length:	1.57m	Maximum pressure:	70Mpa
Weight:	32kg	Deformation resolution:	.005mm

Preliminary data sheet

SERATA GEOMECHANICS, INC.

PROPERTY/STRESSMETER P-200

The second-generation Property/stressmeter P-200, incorporating improvements to the original P-100 design, is a computerized borehole probe capable of determining _in situ_ material properties in complex hard ground and _in situ_ stress state in soft ductile ground. It is specifically designed to measure the properties of fractured and nonfractured rocks, weakness seams, soft yielding media, and other anomalies. The material property coefficient values obtained by the P-200 probe may be used as input for constructing a realistic FEM computer model of the ground.

The cylindrical body of the instrument houses eight penetrometer pistons deployed in two mutually perpendicular orientations in symmetry (Fig. 1). These enable eight penetration tests to be performed simultaneously on the borehole wall. These tests determine point characteristics of the ground at the individual piston contacts. Integration of the point characteristics discloses the true material properties of complex ground, which cannot be characterized accurately by any conventional material testing procedure.

The computerized system automatically collects, analyzes, and graphically displays the data on a laptop computer during the process of measurement (Fig. 2). Monitoring of analyzed data on the screen in real time enables the operator to scrutinize the ground condition for optimization of the measurement. A set of eight tests can be completed in 10 minutes or less, providing an integrated "picture" of the quantitative ground conditions.

In addition to the measurement of material properties _in situ_, the P-200 is also capable of determining stress fields in soft ductile rock media -- particularly salt, potash, coal, mudstone, and hard soil -- for which no conventional method of stress measurement is effective.

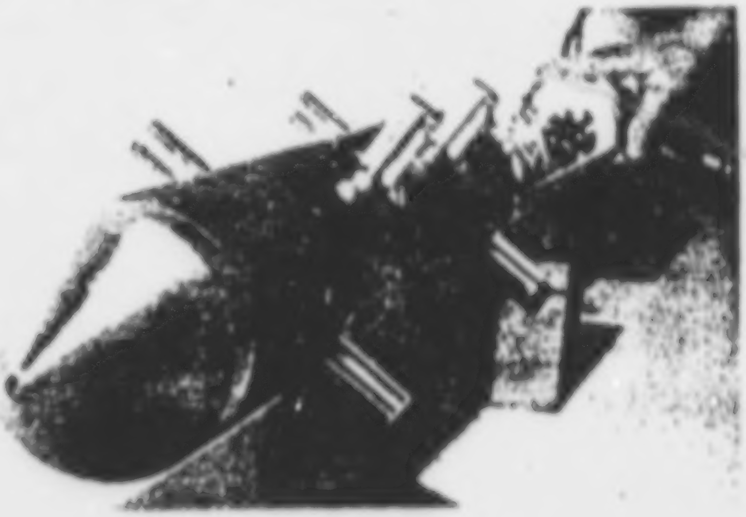


FIG. 1. P-200 probe with pistons extended

Within a borehole drilled in the soft media, the pistons penetrate deepest in the direction of maximum stress and least in the direction of minimum stress. This mechanism is similar to hydrofracturing, in which the failure plane develops in the direction of maximum stress. In the four directions of 45 degrees deviating from the maximum and minimum stress orientation, all the four piston penetrations are equal, dictated by the effect of the mean stress, $\sigma_m = (\sigma_{max} + \sigma_{min})/2$. The different magnitudes of penetration in the maximum and minimum stress directions indicate the magnitude of the shear stress: $\sigma_s = (\sigma_{max} - \sigma_{min})/2$. The rheological finite element program REM® can be applied to determine the stress state and material properties from the measurement.

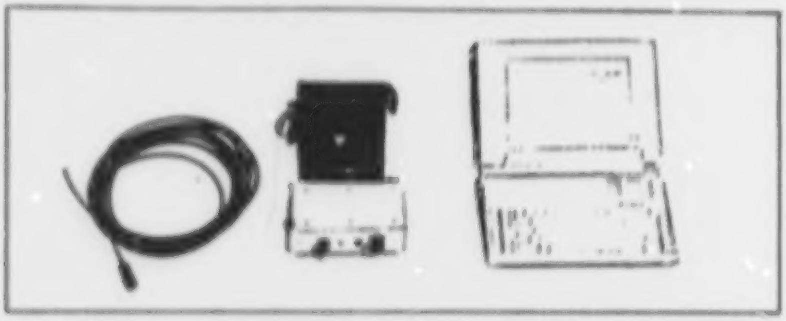


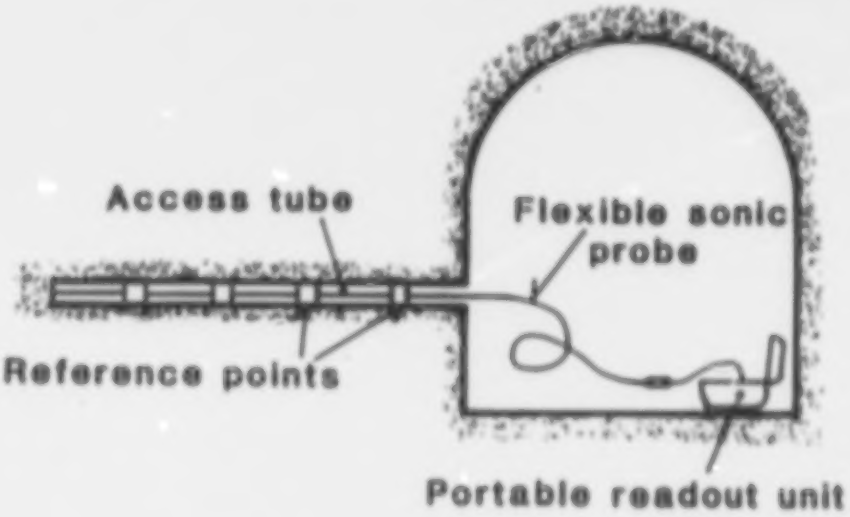
FIG. 2. P-200 accessory package

P-200 Specifications

Probe diameter:	71 mm
Probe length:	2 m
Probe weight:	19 kg
Borehole diameter:	76 ± 1.5 mm
Maximum oil pressure:	700 kgf/cm^2
Maximum piston penetration:	25.4 mm
Pressure accuracy:	0.05% of full scale
Displacement accuracy:	0.001 mm max.
Piston tip loading geometry:	flat or conical

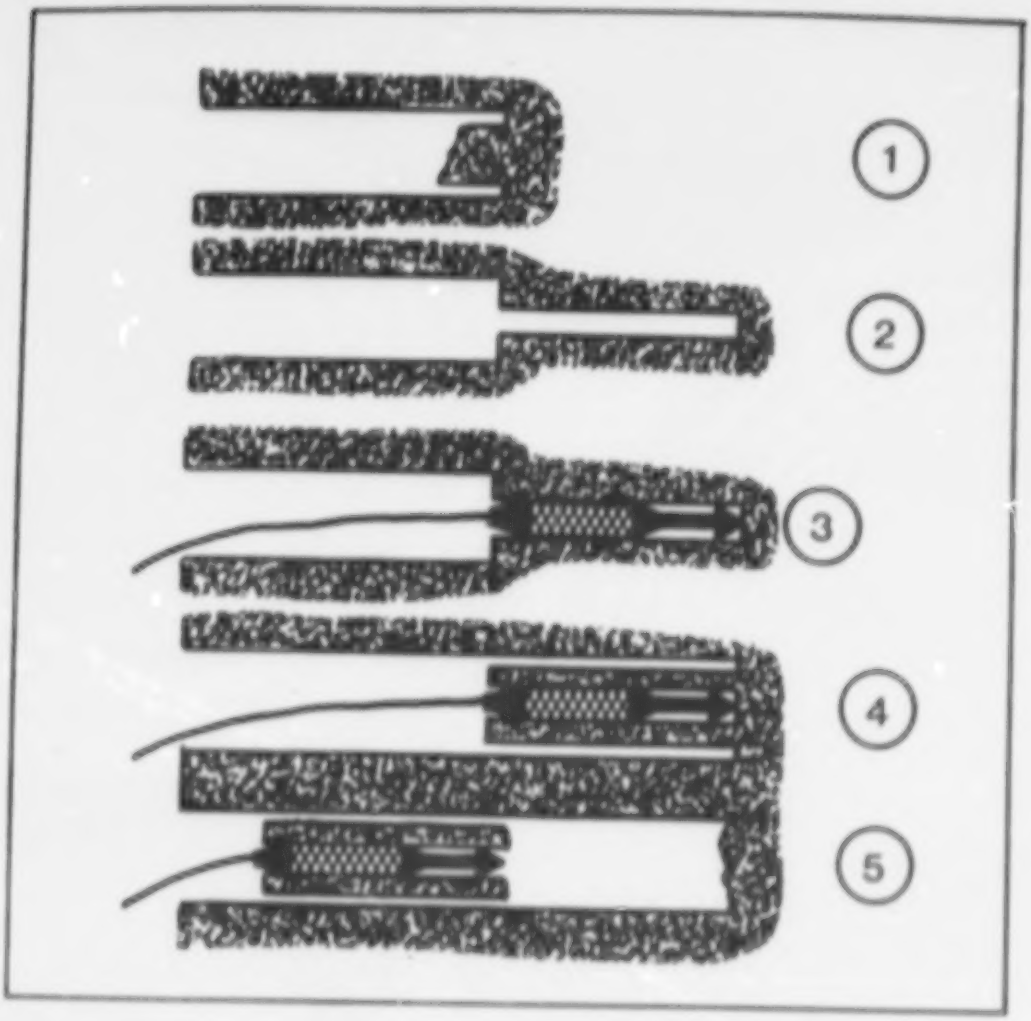
NAME OF INSTRUMENT: IRAD GAGE SONIC EXTENSOMETER

TYPE:	Borehole Extensometer
PRINCIPLE:	Ultrasonic pulses pass down a flexible probe and are 'reflected' back up the probe by the magnet in each reference point. Distances between reference points are found by measuring relative differences in arrival times of reflected pulses
APPLICATIONS:	Measuring axial deformation in a borehole, particularly ahead of advancing faces
ADVANTAGES:	Installation of reference points is quick and simple provided borehole is in good condition Can be used in situations where detailed measurements are required as up to 21 reference points may be installed in 1 hole Large number of sites can be monitored at the same time No bulky equipment (base plates, tensioners, etc) at borehole collar. Can be used in confined spaces or where face will advance Accurate readings can be taken quickly and easily
DISADVANTAGES:	High initial and running costs Depth of readings limited by length of probe to 7 m Equipment needs regular maintenance by skilled electrical technician
COSTS:	Initial: R10 000 — Readout unit (imported from the USA) R7 000 — Flexible probe Running: R25 — each reference point
ACCURACY:	Range: 300 mm dilation, unless hole shears Resolution: 0.2 mm
INSTALLATION:	* 1 technician, 1 helper and 1 driller A BX borehole is drilled to a maximum depth of 7m Fractures, bedding planes, etc are observed with petroscope Reference points are inserted to required depth in borehole and anchored by releasing circlips whose expanded diameter is greater than the borehole Permanent access tube is pushed through centre of each reference point
MONITORING:	* 1 technician Flexible probe is passed down access tube to end of hole, and then connected to the readout unit which gives distances between successive reference points


Access tube
Flexible sonic probe
Reference points
Portable readout unit

NAME OF INSTRUMENT: CSIRO TRIAXIAL HOLLOW INCLUSION STRESS CELL

TYPE:	3-D borehole deformation gauge
PRINCIPLE:	Electrical resistance strain gauges embedded in epoxy body measure strains induced in the rock around the borehole by the relaxation of an annulus of rock during overcoring
APPLICATIONS:	Measuring absolute stress by overcoring or measuring stress changes during long term installation (measured strains converted to stress by numerical solution)
ADVANTAGES:	Provides the complete three dimensional stress state from a single borehole Errors may be accurately estimated as the stress state is overdetermined Continuous read-out of all strain gauges is possible during overcoring, thus providing valuable diagnostic information
DISADVANTAGES:	Experienced personnel required Success is strongly dependent on the quality of the glue bond between instrument and rock Must be installed in competent rock Difficult to use in very hot, dusty or wet conditions Relatively expensive
COSTS:	Initial: R1 500 — per cell (not reusable) R1 500 — installation equipment R2 000 — good quality strain indicator
ACCURACY:	Range: Depends on rock strength Resolution: About 5 microstrains with a good strain indicator
INSTALLATION:	* 1 engineer, 1 technician and an experienced driller NXCU diamond drilled hole is drilled to required depth and bottom of hole flattened (1) EX hole is centred in bottom of hole and extended to required depth (2) EX core is examined for integrity and installation depth chosen Glue is applied to gauges Cell is installed at required depth using a trolley (3) Strain readings are recorded after glue has set Cell is overcored and strain readings taken during overcoring (4) Core containing the cell is recovered and final strain readings are taken (5) Core can be placed in a biaxial cell to determine its properties or these can be measured from lab tests on core specimen Stresses are calculated from strain changes
MONITORING:	* 1 observer Strain changes in the cell are measured periodically at sites where long term stress changes in the rock mass are being monitored


①
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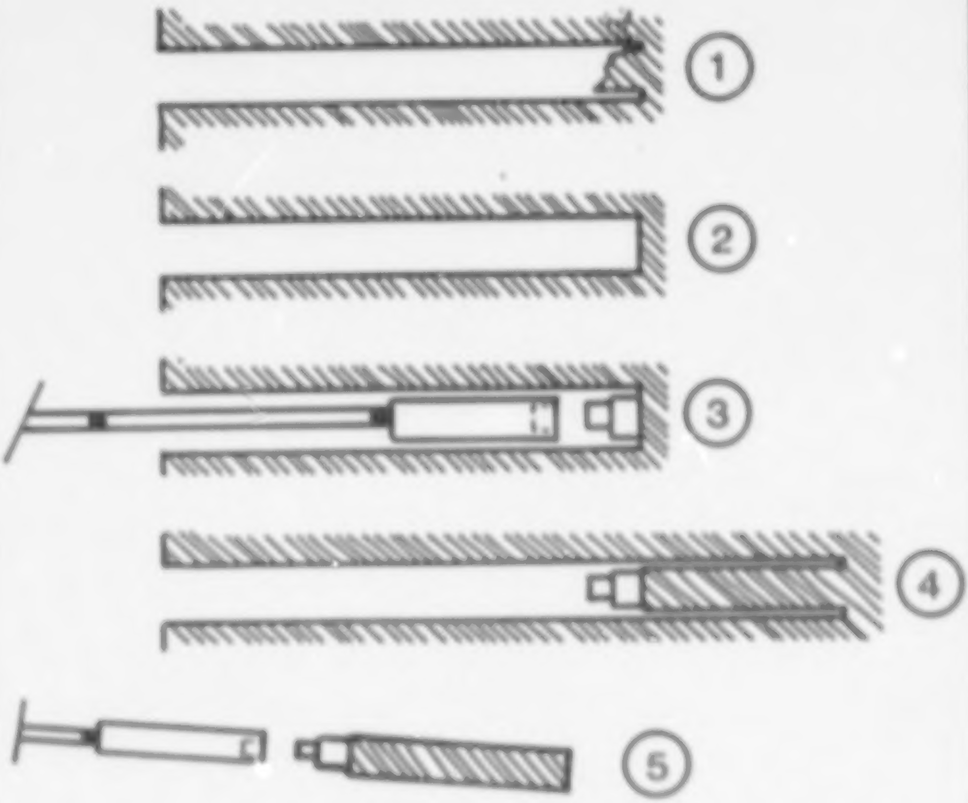
NAME OF INSTRUMENT: CSIR DOORSTOPPER STRAIN CELL

TYPE:	2-D borehole deformation gauge
PRINCIPLE:	Electrical resistance strain gauges mounted on the end of a plastic holder are glued to the end of a borehole and used to measure strain changes in the rock on a long term basis or during overcoring
APPLICATIONS:	Measuring in situ stresses or stress changes
ADVANTAGES:	Relatively inexpensive and fairly easy method of obtaining stresses in 2-D Equipment is robust and mineworthy
DISADVANTAGES:	Some adhesives set too rapidly — this makes it difficult to use in long holes The borehole flattening bits wear out rapidly and must be renewed often The measurement is only two-dimensional; three nonparallel holes are required for a complete stress determination
COSTS:	Initial: R1 500 — installing tool and rods R2 000 — good quality strain indicator R50 — per doorstopper
ACCURACY:	Range: Depends on the rock strength Resolution: 5 microstrains
INSTALLATION:	* 1 experienced technician, 1 assistant and 1 experienced driller BX hole is drilled slightly updip to the required depth (1) End of hole is flattened using special bit (2) Doorstopper is glued to the end of the hole using the special orientating installing tool (3) Zero strain readings are taken Doorstopper is overcored (usually with BX bit) and retrieved (4) Strain readings are taken (5) In situ stress is calculated from the strain readings
MONITORING:	* 1 trained observer Strain changes at a permanent installation are measured periodically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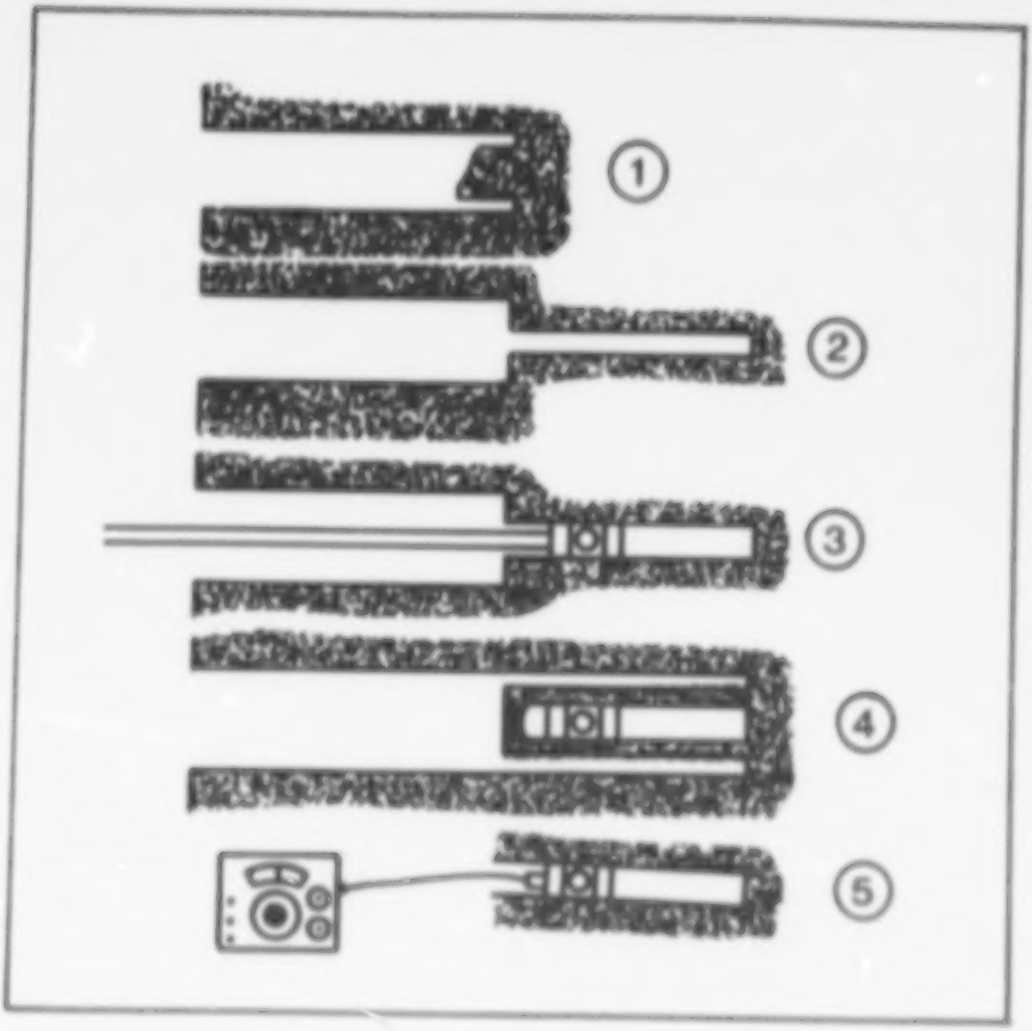
NAME OF INSTRUMENT: CSIR TRIAXIAL STRAIN CELL

TYPE:	3-D borehole deformation gauge
PRINCIPLE:	Electrical resistance strain gauges bonded to the inner surface of a diamond drilled borehole measure strains induced in the rock
APPLICATIONS:	Measuring absolute stress by overcoring. With modification can be used to measure stress changes during long term installation.
ADVANTAGES:	Measures the complete three dimensional stress state from a single borehole Stress state is overdetermined and errors may thus be accurately estimated Dummy gauge is attached directly to the cell Simple and relatively robust Uses similar installation equipment to CSIR doorstopper strain cell Relatively inexpensive
DISADVANTAGES:	Long holes are difficult to instrument, especially where space is limited Successful glueing of the strain gauges to the rock is often a problem, especially in hot and humid conditions Experienced personnel required
COSTS:	Initial: R1 500 — installation equipment R150 — per cell (not reusable) R2 000 — good quality strain indicator
ACCURACY:	Range: Depends on rock strength Resolution: About 5 microstrains with a good strain indicator
INSTALLATION:	* 1 engineer, 1 technician and an experienced driller NXCU diamond drilled hole is drilled to required depth (1) End of hole is flattened and extended using EX stub pilot bit (2) EX hole is extended for about 300mm and core examined for integrity Glue is applied to gauges Triaxial strain cell is inserted into hole and strain gauge pistons pneumatically forced against borehole walls (3) Initial strain readings are taken EX hole is extended for about 300 mm and core examined for integrity Final strain readings are taken (5) Core can be placed in biaxial cell to determine modulus Stresses are calculated from strain changes
MONITORING:	* 1 observer Strain changes at a permanent installation are determined periodically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NAME OF INSTRUMENT: GLÖTZL PRESSURE CELL

TYPE: Uni-directional stress measuring device

PRINCIPLE: Pump is operated until pressure in feed line just equals pressure acting on pressure cell. Pressure delivered in pump equals pressure acting on the cell

APPLICATIONS: Measuring stress in backfill materials, concrete structures, shotcrete linings and rock

ADVANTAGES: Relatively large range

Piping need not necessarily be pressurised continuously

Single pump (required for reading) can service many installations

Transportation underground is easy since pipes can all be connected in situ

DISADVANTAGES: System is expensive

Pump is heavy and pumping action results in pressure fluctuations of up to 0,5 MPa

Taking readings can be time consuming

COSTS:

Initial: R800 to R2 000 — pump
R400 — manifold

Running: R600 — pressure cell (not always recoverable)
R10 — tubing per metre
R50 — frame

ACCURACY: Range: 60 MPa

Resolution: 0,5 MPa

INSTALLATION: ° 1 technician

Pressure cell is placed in required position, using a retaining frame if necessary, and feed and return pipes are connected

Additional lengths of pipe are connected to achieve required distance to reading station

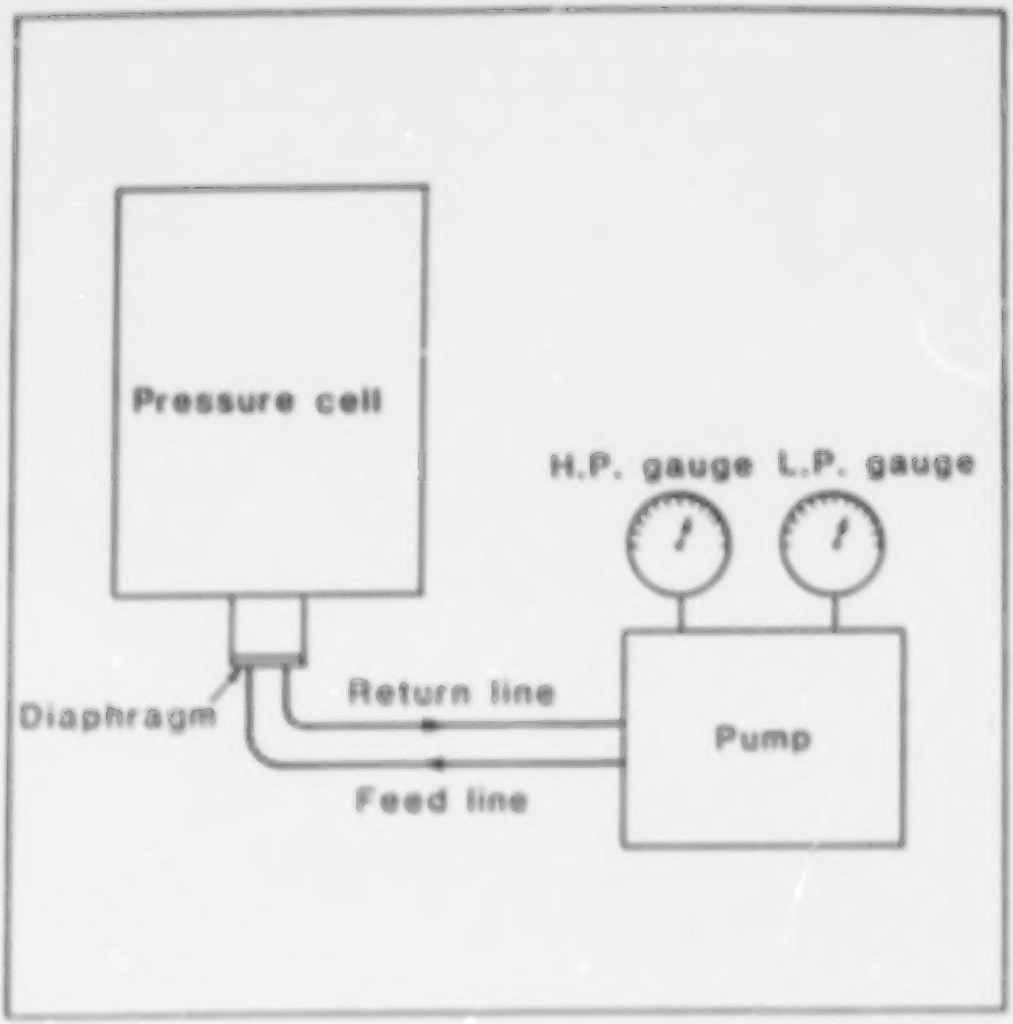
Pipes are connected to pump (single installation) or to manifold (multiple installation)

Feed line is pressurised using pump and hydraulic fluid in the circuit until pressure delivered in the pump equals that in the load cell. Diaphragm then opens allowing a return of fluid to the pump

Highest steady reading is then taken

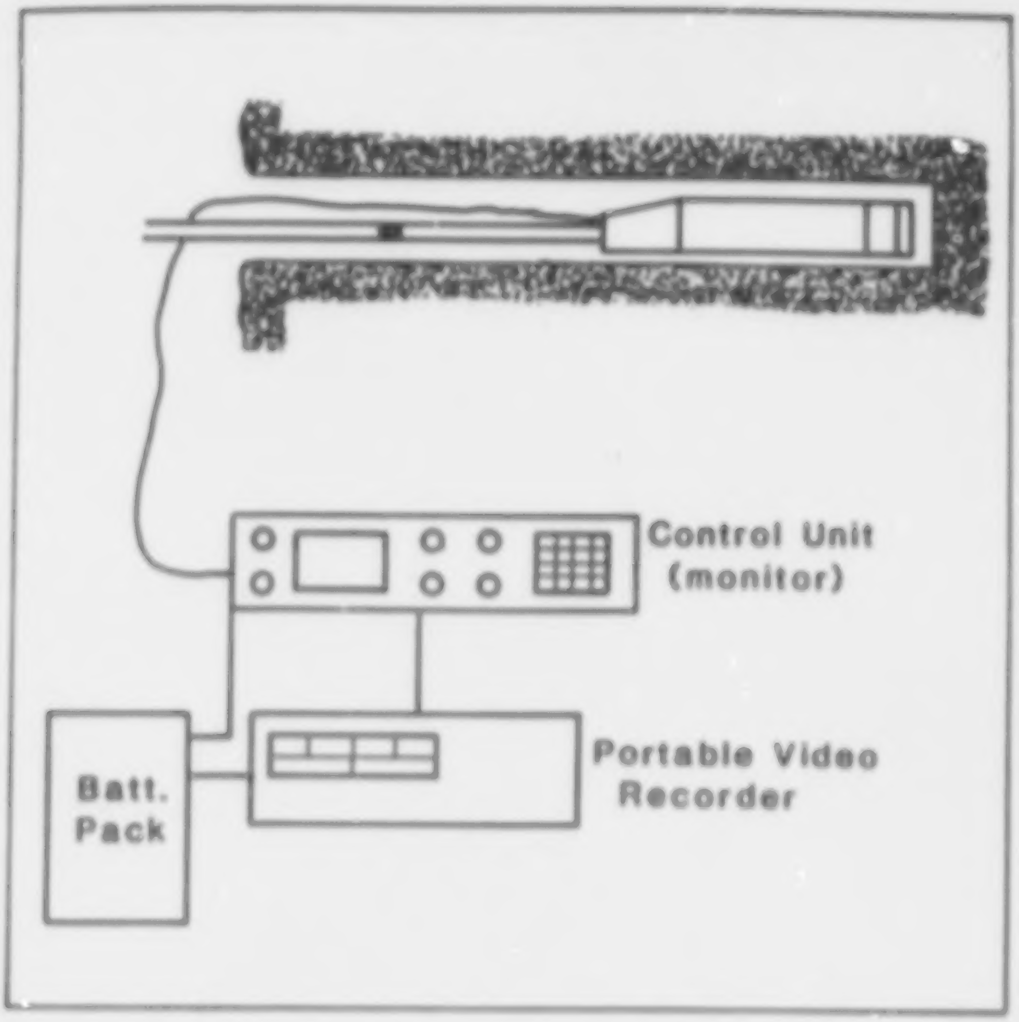
MONITORING: ° 1 technician

Pressure readings are determined periodically


Pressure cell
H.P. gauge
L.P. gauge
Return line
Diaphragm
Pump
Feed line

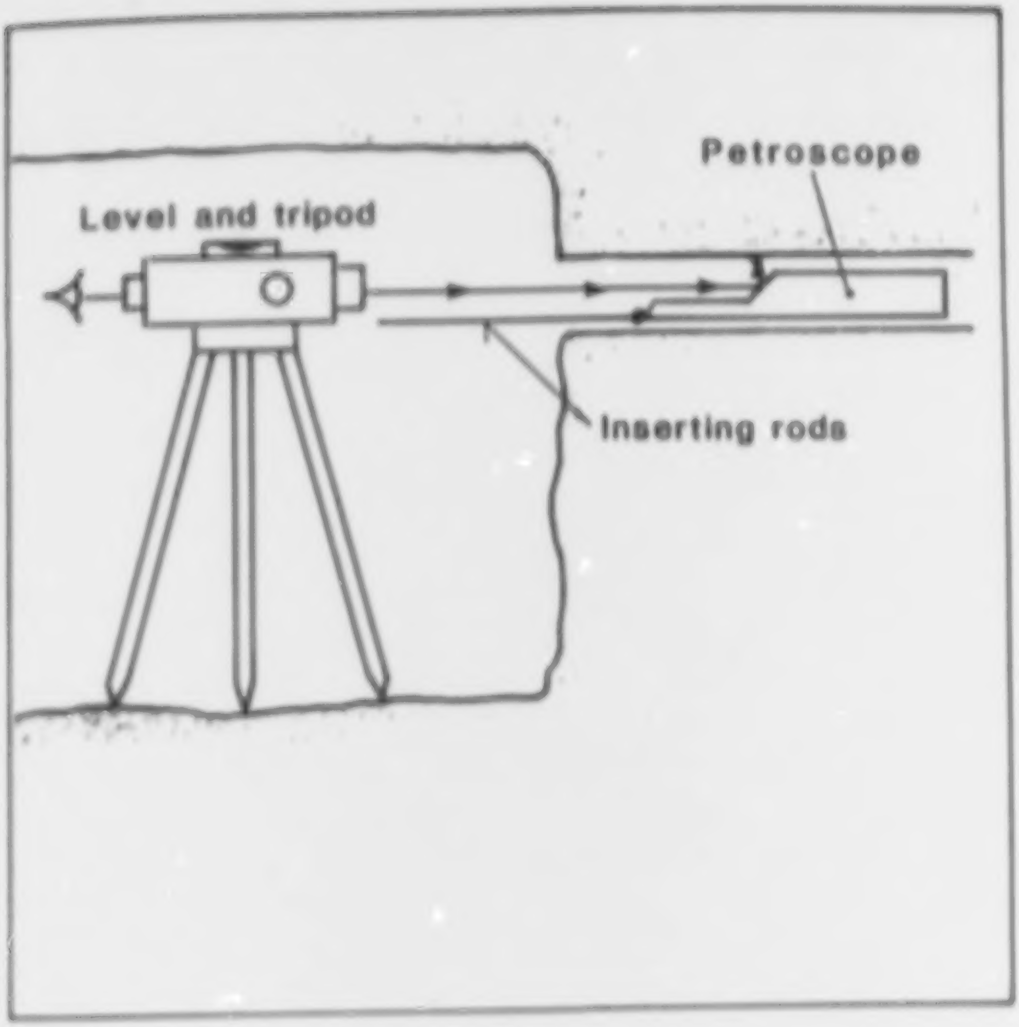
NAME OF INSTRUMENT: REES BOREHOLE CAMERA

TYPE:	Small diameter borehole television camera
PRINCIPLE:	Video camera in borehole transmits images to video recorder via a training cable. Data may also be entered from keyboard on the control unit
APPLICATIONS:	Observing fracturing, spalling etc in boreholes of at least BX size and troubleshooting of problems in pipelines
ADVANTAGES:	Information recorded underground can be taken to surface and processed at leisure Recording provides a permanent record Fish-eye lens gives much better perspective than view from a remote telescope Works under water Can be used at greater depths than can a petroscope
DISADVANTAGES:	Needs to be acclimatized to operating temperature and humidity Equipment is bulky and heavy Expensive
COSTS:	Initial: R40 000 — camera, control unit and recorder
ACCURACY:	Range: Connecting cables 15m long Resolution: Can see fractures less than 1mm wide
INSTALLATION:	* 1 experienced technician and 1 assistant Camera, control unit, video recorder and power source are connected Camera is pushed into borehole
MONITORING:	N/A


Control Unit
(monitor)
Portable Video
Recorder
Batt.
Pack

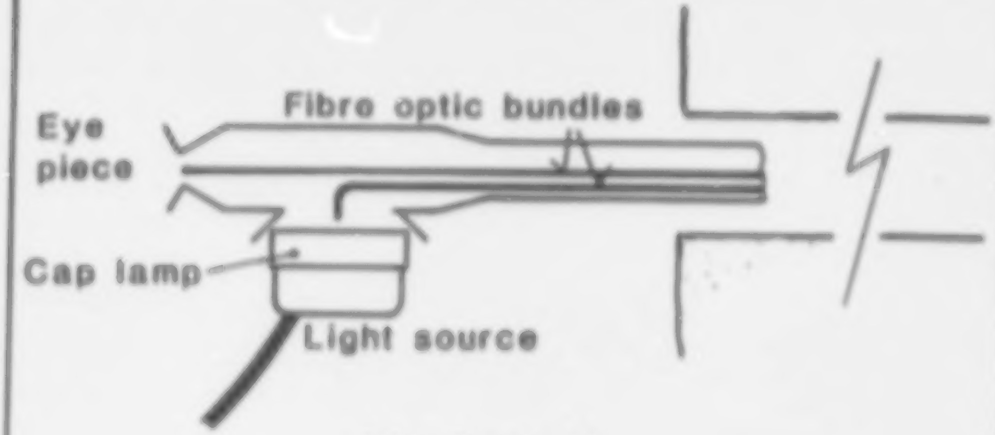
NAME OF INSTRUMENT: PETROSCOPE

TYPE:	Borehole inspection instrument
PRINCIPLE:	An inclined mirror and light source with an onboard battery pack enable the illuminated walls of boreholes to be inspected using a telescope mounted at the borehole collar
APPLICATIONS:	Examining intensity, extent and orientation of fracturing and bedding plane separation in rock mass around any excavation or pillar where depth of fracturing does not exceed 12 m Determining regions of high stress or stress increase by observing spalling Determining optimum position in a borehole for installing extensometer reference points, stress measuring cells, borehole packers etc
ADVANTAGES:	Inexpensive and simple to use Portable; can be carried relatively easily into narrow stopes Can operate to 12 m if borehole is straight. Interpreting image becomes difficult beyond 12 m Intrinsically safe
DISADVANTAGES:	Relies on accurate and consistent interpretation of what is seen in the borehole. Operators must be trained Cannot operate in water filled holes. Water must be pumped out In common with all borescopes, problems arise when hole is badly fractured, spalled, stepped etc
COSTS:	Initial: R1 500 — petroscope R500 — telescope R40 — rods/m R150 — battery & charger Running: Bulbs, batteries and mirrors
ACCURACY:	Range: 12 m Resolution: 0,5 mm
INSTALLATION:	N/A
MONITORING:	* 1 technician and 1 observer Petroscope is inserted into the hole Depth of each fracture, together with its width are recorded Orientation of fracture is recorded by measuring distances along the borehole to top, left and right positions of fracture as it transects borehole Intensity, orientation and depth of spalling is also noted

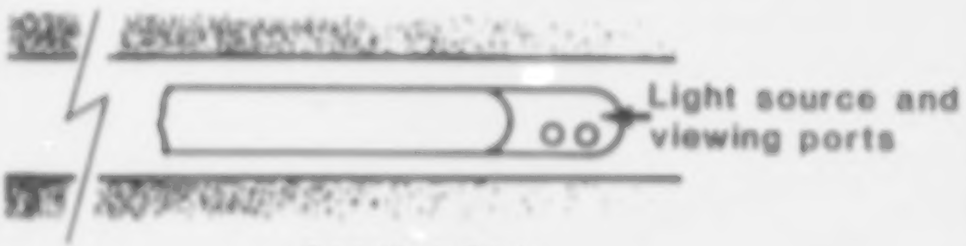

Petroscope
Level and tripod
Inserting rods

NAME OF INSTRUMENT: STRATASCOPE

TYPE:	Flexible fibre optic borehole scope
PRINCIPLE:	Light from an external source is transmitted to end of Stratascope via a fibre optic bundle and side of the hole is illuminated. Image of illuminated area is transmitted back along instrument to eyepiece for viewing
APPLICATIONS:	Inspecting boreholes for fractures, slips, spalling etc.
ADVANTAGES:	Intrinsically safe; can be used in collieries Can be obtained in varying lengths Can operate in holes that are not straight
DISADVANTAGES:	Expensive Area illuminated is quite small. Image interpretation difficult, especially in small boreholes Vision easily obscured at reading end by dirt in hole Special cap lamp is needed to ensure high quality illumination of hole
COSTS:	Initial: R10 000 — instrument (imported from USA) R300 — cap lamp
ACCURACY:	Range: Variable Resolution: 0,5 mm
INSTALLATION:	* 1 technician Cap lamp acting as a light source is attached to Stratascope Instrument is then inserted into borehole to be inspected
MONITORING:	N/A


Fibre optic bundles
Eye piece
Cap lamp
Light source

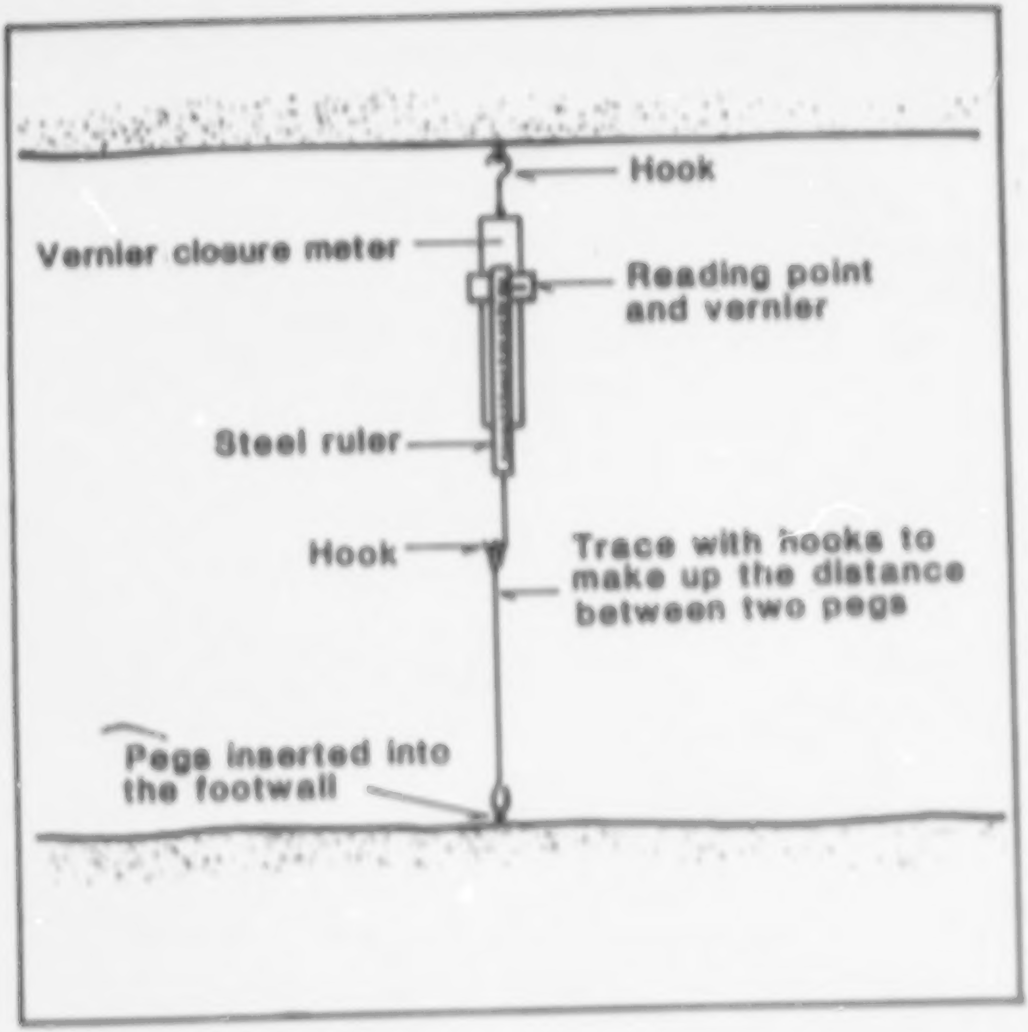
Viewing End


Light source and viewing ports

Reading End

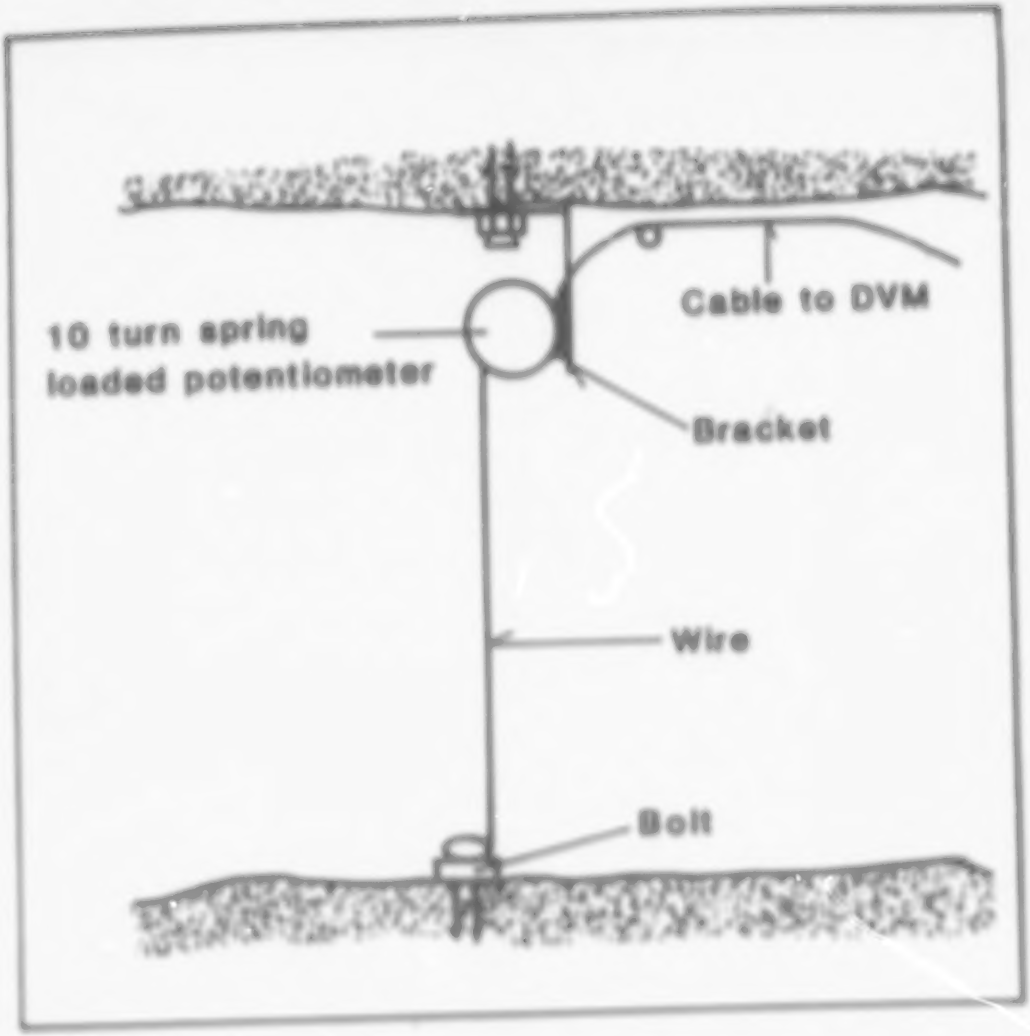
NAME OF INSTRUMENT: VERNIER CLOSURE METER

TYPE:	Spring Loaded Closure Meter
PRINCIPLE:	Relative displacements between bolts installed in excavations are measured using lengths of wire trace and a spring loaded vernier
APPLICATIONS:	Measuring convergence in excavations in relatively confined areas, or over short distances where great accuracy is not required
ADVANTAGES:	Low cost Robust and easy to use Easily carried
DISADVANTAGES:	Not as accurate as Tape Extensometers Does not give absolute distance between measuring points, although this can be determined using a tape Cannot be used for measuring convergences over distances greater than 4 m because of the soft spring in the meter Less accurate for measuring horizontal convergence
COSTS:	Initial: R220 — vernier closure meter Running: 0,2 m — 4 m
ACCURACY:	Resolution: 0,5 mm — vertical 1,0 mm — horizontal
INSTALLATION:	* 1 technician and 1 assistant Holes are drilled in rock at required locations Measuring bolts are secured in holes either with epoxy or using an expansion device
MONITORING:	* 1 observer Changes in distance between bolts are measured by attaching meter to one bolt, and suitable length of wire trace to another Reading is then taken on vernier


Hook
Vernier closure meter
Reading point and vernier
Steel ruler
Hook
Trace with hooks to make up the distance between two pegs
Pegs inserted into the footwall

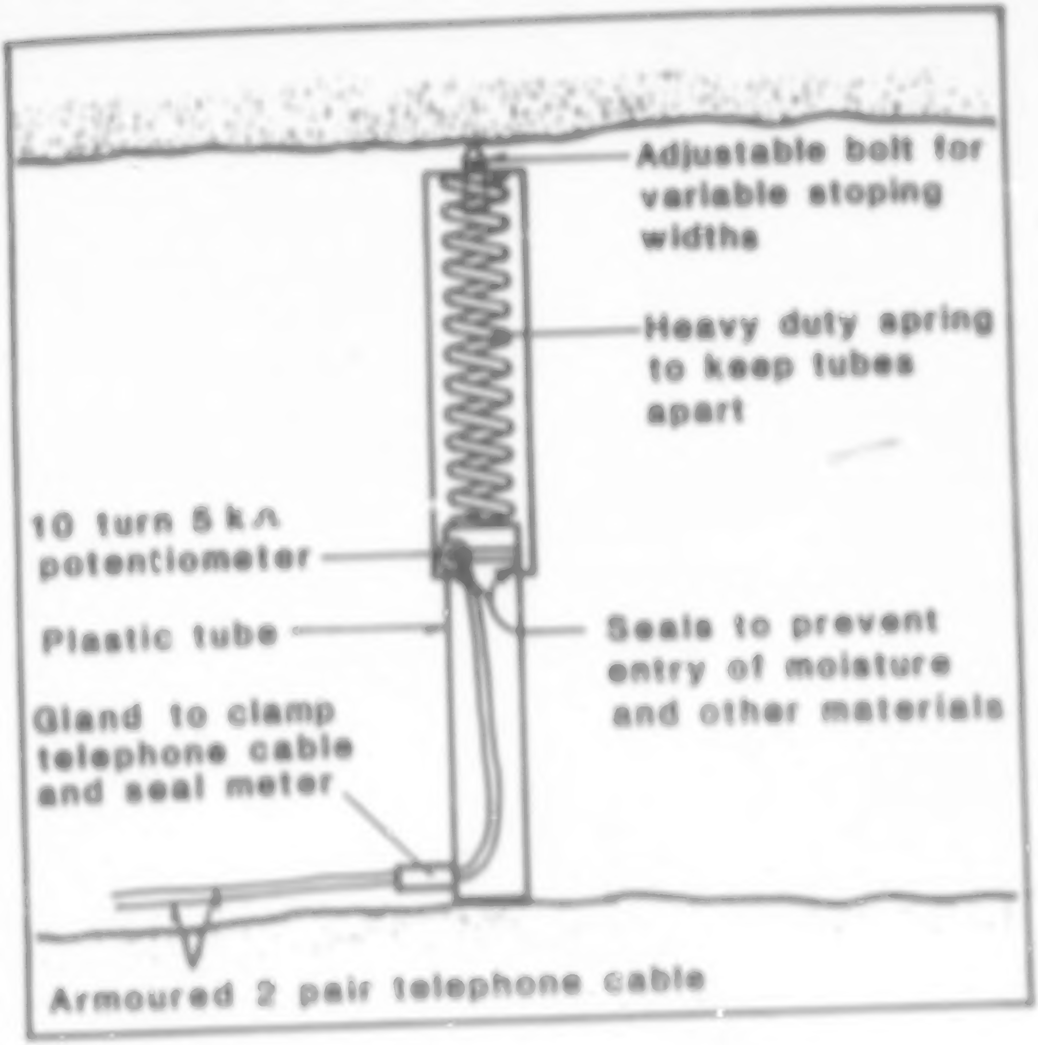
NAME OF INSTRUMENT: COM STOPE CLOSURE METER

TYPE:	Spring loaded potentiometric closure meter
PRINCIPLE:	Movement between hangingwall and footwall of an excavation is indicated by change in electrical output from 10 turn spring loaded potentiometer
APPLICATIONS:	Measuring closure in stopes over long periods
ADVANTAGES:	Good long term stability if protected Fairly accurate Can be logged remotely using datalogger Withstands underground atmosphere well Digital readings using DVM are not subjective Measurements can be taken by unskilled personnel
DISADVANTAGES:	Needs adequate protection — not blast resistant Fairly expensive
COSTS:	Initial: R800 — closure meter R300 — good quality multimeter e.g. Fluke Running: R4 each bolt
ACCURACY:	Range: 0,3 m to 1,2 m Resolution: 0.001 of full scale reading
INSTALLATION:	* 1 technician and 1 assistant Anchor bolts are installed in roof and floor Closure meter is attached to roof bolt and a spring wire is attached to floor bolt Resistance of potentiometer is read using DVM
MONITORING:	* 1 experienced observer Potentiometer reading is determined periodically. Resistance is converted to closure using calibration curves or equations


Cable to DVM
10 turn spring loaded potentiometer
Bracket
Wire
Bolt

NAME OF INSTRUMENT: COM CLOSURE METER

TYPE:	Potentiometric Closure Measuring Device
PRINCIPLE:	A plastic tube is positioned inside a larger tube with a strong spring maintaining tension between the two. As convergence of excavation occurs, smaller tube moves into larger one, rotating a potentiometer which is calibrated to give displacement
APPLICATIONS:	Remote monitoring of convergence in stopes May also be used in areas supported by backfill
ADVANTAGES:	Relatively inexpensive and simple to construct May be used in areas where access after installation is difficult or dangerous
DISADVANTAGES:	Convergence range limited to half the width of excavation Can only tolerate limited horizontal compression Potentiometers must not be sensitive to humidity
COSTS:	Initial: R400 — multimeter for monitoring Running: R320 — convergence meter R2 — cable/m
ACCURACY:	Range: Limited to half width of excavation Resolution: 1 mm
INSTALLATION:	* 1 technician and 1 assistant Instrument is placed between hangingwall and footwall at desired location. An armoured 2 pair cable is run out to position where there will always be easy access
MONITORING:	* 1 observer Resistance of potentiometer is read using a multimeter and converted to displacement using calibration data


Adjustable bolt for variable stoping widths
Heavy duty spring to keep tubes apart
10 turn 5 kΩ potentiometer
Plastic tube
Seals to prevent entry of moisture and other materials
Gland to clamp telephone cable and seal meter
Armoured 2 pair telephone cable

APPENDIX J

AN INVESTIGATION INTO THE DESIGN OF TIGHT PACKS IN HARD ROCK MINE BACKFILL

BY

DEPARTMENT OF MINING AND METALLURGICAL ENGINEERING
McGILL UNIVERSITY

AN INVESTIGATION INTO DESIGN OF TIGHT PACKS IN HARD ROCK MINE BACKFILL

Department of Mining and Metallurgical Engineering

McGill University

Montreal, Quebec

Canada, H3A 2A7

EXECUTIVE SUMMARY

This study was primarily conducted to evaluate the potential of anhydrite as a binder when mixed with mine tailing for underground hard rock mining.

Anhydrite ($CaSO_4$) will in presence of water form dihydrite ($CaSO_4$, $2H_2O$) a strong solid material. The transformation is far too lengthy for practical mining application unless an accelerator is introduced.

In order to highlight the potential of anhydrite as a binder, series of laboratory tests were conducted to determine the physical and mechanical properties of different mixes of anhydrite, tailing, accelerator and water, at different curing time.

The details of the sample preparation and testing techniques are given in the report. Summary of the results are given the following tables (Tables 7 and 21):

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS

(AS OF 1985 PRICES)

Pack Type	Material Costs (CAN$/m^3 of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Aquapak	Aquabent	50	12.50		0.52	1.30	4.00	5.00
	Aquacem	450	95.86	108.36				
Thyssen	Coal	588		0.45	0.48	0.95	1.34	
	Flowmat	16	2.16					
	Packbind	298	64.44	100.17				
Warbrat	Coal	588	33.57		0.07	2.40	3.10	4.90
	Flowmat	16	2.16					
	O.P.C. (Bulk)	298	23.53					
	TEA 51	30	24.03	83.29				
Takpak	Takbent	182	40.85		0.67	2.50	4.0	4.2
	Tekcem	182	57.95	98.80				
Astrapak	Astrabent	183	37.75		0.5	1.4	4.4	4.5
	Astracem	183	52.96	90.71				

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MANOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/m^3 of pack)							
	Materials		Costs excluding transport and crib bag		Uniaxial Compressive Strength (MPa)			
	Name	Wieght (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Flashpak	Fly Ash	921	27.63		0.02	0.9	4.1	9.3
	O.p.c. (Bulk)	255	41.31					
	$FeSO_4$	100	22.50					
	Fulbent	18	5.94	97.38				
	Fly Ash	930	30.63		0.40	1.6	4.4	8.2
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	65	24.62					
	Fulbent	95	31.67	128.23				
	Fly Ash	930	30.63					
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	43	16.29	88.23				
Preblend	Limestone$^+$	2000	73.59		0.78	1.53	8.80	9.80
	Rapid Setting							
	Cement (Bulk)			73.59				

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/$m^3$ of pack)							
	Materials		Costs excluding transport and crib bag		Uniaxial Compressive Strength (MPa)			
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Anpak	Synthetic Anhydrite (Bulk)	2000	39.79		0.26	6.1	21.0	28.0
	Accelerator	20	8.65	48.44				
	Natural Anhydrite (Bulk)	2000	19.90		0.05	0.8	14.5	20.3
	Accelerator	56	24.22	44.12				

Table 21 - UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS FOR THE ANTAIL.

Serial No.	Mesh Size	Curing Days	Anhydrite Parts.	Tailings (Parts)	Water Parts.	FeSO4 (Parts)	Density (gr. cu.cm.)	U.C.S. MPa	Strain %
1	-200	7	50	50	25	2.8	2.130	3.728	0.4
2	-200	7	25	75	25	2.8	2.151	1.570	0.2
3	-200	7	50	50	30	2.8	2.077	2.943	0.2
4	-200	7	25	75	30	2.8	1.962	0.294	0.3
5	-200	7	50	50	35	2.8	2.013	2.158	0.5
6	-200	7	25	75	35	2.8	2.091	0.392	0.2
7	-200	7	50	50	25	3.6	2.031	3.139	0.3
8	-200	7	25	75	25	3.6	2.092	1.472	0.3
9	-200	7	50	50	30	3.6	2.022	2.747	0.2
10	-50	7	50	50	32	2.8	1.924	2.256	0.3
11	-50	7	25	75	32	2.8	1.898	0.491	0.1
12	-50	7	50	50	37	2.8	1.834	0.589	0.4
13	-50	7	25	75	37	2.8	1.933	0.294	0.4
14	-50	7	50	50	42	2.8	1.835	0.491	0.1
15	-50	7	50	50	32	3.6	1.938	1.472	0.6
16	-50	7	50	50	37	3.6	1.898	1.177	0.5
17	-50	7	25	75	37	3.6	1.958	0.687	0.4
18	-50	7	50	50	42	3.6	1.842	0.491	0.4
19	-100	7	100	0	25	2.8	2.095	16.579	1.3
20	-100	7	75	25	25	2.8	2.066	16.187	0.9
21	-100	7	50	50	25	2.8	2.046	11.380	0.6
22	-100	7	25	75	25	2.8	2.035	2.060	0.7
23	-100	7	100	0	30	2.8	2.011	16.481	1.2
24	-100	7	75	25	30	2.8	1.963	15.696	0.9
25	-100	7	50	50	30	2.8	1.992	9.516	0.7
26	-100	7	25	75	30	2.8	1.901	2.060	0.7
27	-100	7	100	0	35	2.8	1.942	16.187	1.3
28	-100	7	75	25	35	2.8	1.945	15.500	1.1
29	-100	7	50	50	35	2.8	1.978	8.044	0.8
30	-100	7	25	75	35	2.8	2.019	1.364	0.5
31	-100	7	100	0	25	3.6	2.090	16.677	1.2
32	-100	7	75	25	25	3.6	2.075	16.481	0.9
33	-100	7	50	50	25	3.6	2.048	9.320	0.5
34	-100	7	25	75	25	3.6	1.958	1.364	1.0
35	-100	7	100	0	30	3.6	2.029	16.579	0.7
36	-100	7	75	25	30	3.6	2.002	15.696	0.6
37	-100	7	50	50	30	3.6	1.996	7.161	0.7
38	-100	7	25	75	30	3.6	2.030	1.766	0.4
39	-100	7	100	0	35	3.6	1.960	16.088	0.7
40	-100	7	75	25	35	3.6	1.960	14.715	0.8
41	-100	7	50	50	35	3.6	1.995	5.690	0.5
42	-100	7	25	75	35	3.6	2.010	1.472	0.4
43	-200	7	50	50	25	2.8	1.956	6.867	0.4
44	-200	7	25	75	25	2.8	1.961	2.551	0.4
45	-200	7	50	50	30	2.8	1.881	6.377	0.5

T 21

Serial No.	Mesh Size	Curing Days	Anhydrite (Parts)	Tailings (Parts)	Water (Parts)	FeSO4 (Parts)	Density gr./cu.cm.	U.C.S. (MPa)	Strain (%)
46	-200	7	25	75	30	2.8	1.776	1.766	0.3
47	-200	7	50	50	35	2.8	1.899	4.737	0.2
48	-200	7	25	75	35	2.8	1.925	2.354	0.6
49	-200	7	50	50	25	3.6	1.967	17.168	1.3
50	-200	7	25	75	25	3.6	1.963	4.611	1.2
51	-200	7	50	50	30	3.6	1.893	5.575	0.5
52	-200	7	25	75	30	3.6	1.973	2.060	0.2
53	-200	7	50	50	35	3.6	1.979	13.578	0.9
54	-200	7	25	75	35	3.6	1.973	1.864	0.7
55	-20	14	100	0	32	2.8	1.725	1.354	1.0
56	-20	14	75	25	32	2.8	1.858	4.120	0.6
57	-20	14	50	50	32	2.8	1.863	3.335	0.3
58	-20	14	25	75	32	2.8	[illegible]	0.392	0.2
59	-20	14	100	0	37	2.8	[illegible]	3.728	0.9
60	-20	14	75	25	37	2.8	1.810	3.630	0.4
61	-20	14	50	50	37	2.8	1.786	0.589	0.3
62	-20	14	25	75	37	2.8	1.810	0.491	0.2
63	-20	14	75	25	42	2.8	1.652	1.177	0.4
64	-20	14	50	50	42	2.8	1.708	1.373	0.6
65	-20	14	100	0	32	3.6	1.777	1.275	0.5
66	-20	14	75	25	32	3.6	1.910	7.358	0.6
67	-20	14	50	50	32	3.6	1.842	3.924	0.4
68	-20	14	25	75	32	3.6	1.727	0.196	0.2
69	-20	14	100	0	37	3.6	1.686	2.649	0.2
70	-20	14	75	25	37	3.6	1.736	1.177	0.3
71	-20	14	50	50	37	3.6	1.766	2.649	0.4
72	-20	14	100	0	42	3.6	1.507	1.079	0.6
73	-20	14	75	25	42	3.6	1.705	0.981	0.5
74	-20	14	50	50	42	3.6	1.783	1.864	0.5
75	-50	14	50	50	32	2.8	1.816	6.377	0.7
76	-50	14	25	75	32	2.8	1.759	1.079	0.5
77	-50	14	50	50	37	2.8	1.675	2.354	0.5
78	-50	14	25	75	37	2.8	1.732	0.589	0.4
79	-50	14	50	50	42	2.8	1.749	2.747	0.4
80	-50	14	50	50	32	3.6	1.827	9.712	0.7
81	-50	14	25	75	32	3.6	1.812	2.256	0.5
82	-50	14	50	50	37	3.6	1.738	5.886	0.5
83	-50	14	50	50	42	3.6	1.617	3.728	0.8
84	-50	14	25	75	42	3.6	1.724	1.079	0.5
85	-20	28	100	0	32	2.8	1.732	6.867	0.7
86	-20	28	75	25	32	2.8	1.882	16.677	0.5
87	-20	28	50	50	32	2.8	1.794	7.259	0.8
88	-20	28	25	75	32	2.8	1.748	1.766	0.7
89	-20	28	100	0	37	2.8	1.665	5.788	1.0
90	-20	28	75	25	37	2.8	1.739	8.437	0.8

T21

Serial No.	Mesh Size	Curing Days	Anhydrite Parts	Tailings Parts	Water Parts	FeSO4 Parts	Density (gm/cu.cm.)	U.C.S. (MPa)	Strain (%)
91	-20	28	50	50	37	2.8	1.712	5.199	0.6
92	-20	28	100	0	42	2.8	1.668	2.845	0.7
93	-20	28	75	25	42	2.8	1.812	5.592	0.7
94	-20	28	50	50	42	2.8	1.725	5.795	1.0
95	-20	28	25	75	42	2.8	1.792	0.883	0.5
96	-20	28	100	0	32	3.6	1.711	3.826	0.7
97	-20	28	75	25	32	3.6	1.793	7.554	1.0
98	-20	28	50	50	32	3.6	1.774	5.984	0.7
99	-20	28	25	75	32	3.6	1.721	1.962	0.5
100	-20	28	100	0	37	3.6	1.677	3.728	0.8
101	-20	28	75	25	37	3.6	1.755	6.377	0.9
102	-20	28	50	50	37	3.6	1.745	3.630	0.9
103	-20	28	25	75	37	3.6	1.714	0.687	0.3
104	-20	28	100	0	42	3.6	1.562	2.158	0.6
105	-20	28	75	25	42	3.6	1.682	5.396	1.0
106	-20	28	50	50	42	3.6	1.729	5.592	1.3
107	-50	28	100	0	32	2.8	1.688	15.696	1.0
108	-50	28	75	25	32	2.8	1.657	15.206	0.9
109	-50	28	50	50	32	2.8	1.635	9.516	0.9
110	-50	28	25	75	32	2.8	1.693	3.041	0.9
111	-50	28	100	0	37	2.8	1.698	11.282	1.0
112	-50	28	75	25	37	2.8	1.624	10.889	0.9
113	-50	28	50	50	37	2.8	1.610	8.927	0.8
114	-50	28	25	75	37	2.8	1.663	2.256	0.6
115	-50	28	100	0	42	2.8	1.599	11.282	0.5
116	-50	28	75	25	42	2.8	1.589	9.123	0.8
117	-50	28	50	50	42	2.8	1.567	5.101	0.3
118	-50	28	25	75	42	2.8	1.591	1.668	0.3
119	-50	28	100	0	32	3.6	1.700	15.990	1.2
120	-50	28	75	25	32	3.6	1.668	12.555	1.1
121	-50	28	50	50	32	3.6	1.667	6.377	0.8
122	-50	28	25	75	32	3.6	1.680	3.237	0.7
123	-50	28	100	0	37	3.6	1.624	9.418	0.9
124	-50	28	75	25	37	3.6	1.578	11.183	0.6
125	-50	28	50	50	37	3.6	1.705	3.731	0.3
126	-50	28	25	75	37	3.6	1.684	2.474	0.5
127	-50	28	100	0	42	3.6	1.687	14.519	0.5
128	-50	28	75	25	42	3.6	1.540	9.516	0.9
129	-50	28	50	50	42	3.6	1.594	7.259	0.6
130	-50	28	25	75	42	3.6	1.603	1.364	0.5
131	-100	28	100	0	25	2.8	2.003	15.392	1.0
132	-100	28	75	25	25	2.8	1.932	16.579	0.7
133	-100	28	50	50	25	2.8	1.925	15.892	0.7
134	-100	28	25	75	25	2.8	1.849	5.690	0.4
135	-100	28	100	0	30	2.8	1.852	16.579	1.0

T22

Serial No.	Mesh Size	Curing Days	Anhydrite Parts	Tailings Parts	Water Parts	FeSO4 Parts	Density (gr/cu.cm.)	U.C.S. MPa	Strain (%)
136	-100	28	75	25	30	2.8	1.823	15.451	0.3
137	-100	28	50	50	30	2.8	1.870	15.794	0.5
138	-100	28	25	75	30	2.8	1.827	7.703	0.5
139	-100	28	100	0	35	2.8	1.817	17.168	0.6
140	-100	28	75	25	35	2.8	1.795	17.168	0.6
141	-100	28	50	50	35	2.8	1.852	16.187	0.8
142	-100	28	25	75	35	2.8	1.755	7.650	0.3
143	-100	28	100	0	25	3.6	1.927	15.971	2.3
144	-100	28	75	25	25	3.6	1.941	16.677	1.5
145	-100	28	50	50	25	3.6	1.894	16.431	0.7
146	-100	28	25	75	25	3.6	1.908	7.358	0.4
147	-100	28	100	0	30	3.6	1.930	16.677	1.0
148	-100	28	75	25	30	3.6	1.856	17.364	0.8
149	-100	28	50	50	30	3.6	1.808	15.392	0.7
150	-100	28	25	75	30	3.6	1.789	3.302	0.7
151	-100	28	100	0	35	3.6	1.816	16.775	1.5
152	-100	28	75	25	35	3.6	1.828	14.911	0.9
153	-100	28	50	50	35	3.6	1.831	13.440	0.9
154	-100	28	25	75	35	3.6	1.832	1.766	0.4
155	-200	28	100	0	25	2.8	1.856	12.949	2.1
156	-200	28	75	25	25	2.8	1.916	13.342	1.8
157	-200	28	50	50	25	2.8	1.872	11.576	1.0
158	-200	28	25	75	25	2.8	1.898	5.671	0.7
159	-200	28	100	0	30	2.8	1.841	15.481	0.8
160	-200	28	75	25	30	2.8	1.724	15.990	0.8
161	-200	28	50	50	30	2.8	1.744	15.714	0.7
162	-200	28	25	75	30	2.8	1.837	5.229	0.7
163	-200	28	100	0	35	2.8	1.712	12.447	0.4
164	-200	28	75	25	35	2.8	1.712	17.452	0.5
165	-200	28	50	50	35	2.8	1.807	13.745	0.3
166	-200	28	25	75	35	2.8	1.772	9.223	0.8
167	-200	28	100	0	25	3.6	1.746	17.755	0.7
168	-200	28	75	25	25	3.6	1.851	17.560	0.8
169	-200	28	50	50	25	3.6	1.841	17.560	0.7
170	-200	28	25	75	25	3.6	1.811	9.516	0.4
171	-200	28	100	0	30	3.6	1.778	15.990	0.7
172	-200	28	75	25	30	3.6	1.721	15.794	0.4
173	-200	28	50	50	30	3.6	1.794	16.383	0.7
174	-200	28	25	75	30	3.6	1.803	7.259	0.4
175	-200	28	100	0	35	3.6	1.722	16.971	1.1
176	-200	28	75	25	35	3.6	1.694	16.775	0.8
177	-200	28	50	50	35	3.6	1.795	16.677	0.5
178	-200	28	25	75	35	3.6	1.762	5.396	0.3

The results of this study have been compared with other monolithic materials, i.e. mixtures of anhydrite, flyash and mine tailing.

It was found that the pumpable mixtures which provide the best results from the point of properties and cost effectiveness are as follows:

1 - ANTAIL - by weight ratio of:

-50 to +100 mesh : Tailing : Accelerator : Water
Natural Anhydrite

50 : 50 : 2.8 : 25

Density = 1600 kg/m^3, after 28 days of curing
U.C.S. = 8.9 MPa
E = 1.1 GPa.

2 - ANTAIL - by weight ratio of:

-100 mesh Natural : Tailing : Accelerator : Water
Anhydrite

25 : 75 : 2.8 : 25

Density = 1894 kg/m^3, after 28 days of curing
U.C.S. = 5.7 MPa
E = 1.4 GPa.

3 - ANTAIL - by weight ratio of:

-100 mesh Nat. : Tailing : Accelerator : Water
Anhydrite

50 : 50 : 2.8 : 25

Density = 1900 kg/m^3, after 28 days of curing
U.C.S. = 15.9 MPa
E = 1.9 GPa.

It is recommended that adequate tests under the triaxial and field (or pilot scale) conditions be conducted to compare and validate the performance and economical effectiveness of the above noted results with that actually will be experienced in the mines.

It is also recommended that when such material are used and placed in the field, appropriate field samples to be taken and tested for comparison studies and quality control together with effect of mine environment and placement technique.

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1 - INTRODUCTION

Hydraulically transportable, quick setting materials have long history of successful utilisation as packs in soft rock mining. They have been predominantly used in high stress areas of Coal Mines as "Monolithic Packs" with various commercial names such as: Aquapak, Tekpak, Astrapak, Anpak and Flashpak. Their major ingredients are: high alumina cement and bentonite; or anydrite; or flyash, which by introduction of water and aceelerator (or retarder) form a quick setting slurry. It is then pumped to the point where the pack is to be set.

Since this type of packs have not been used in hard rock mining, most of the references made in this investigation is due to experiences gained on soft rocks.

Advantages of the above noted pack type over the conventional packs and backfills are as follows:

a - Quicker curing and setting upon placement, they develop early strenght, thereby reducing the closure or deformation of the excavated area.

b - Higher and a more uniform strength development which can be adjusted or designed to suite the ground condition to be supported.

c - Greater resistance against water.

d - Lower porosity and permeability to water and air.

e - Simpler and quicker to prepare and inplace.

f - They can be used in conjunction with the mined waste material or backfills.

Potential utilisation of the monolithic packs in hard rock mining is as follows:

A - As "Tight Pack", to be pumped on top of the backfill. With this action, the gap between the backfill and the roof of the mined stope is filled quickly. Thereby providing an early, uniform and stronger support for the top of the stope, before excessive deformation or failure occur.

B - As a higher strength material than the backfill to be placed arround the bulkheads, where excessive stresses are encountered.

C - As a quick setting cementious material to stop or reduce sloughing of localised stope walls.

This investigation is concerned with determination of physical properties of quick setting tight packs to be utilised in hard rock mining. However, a resume on the types and system of the monolithic packs related to both soft and hard rock mining is enclosed here.

2 - TYPES OF THE MONOLITHIC PACKS

2.1 - AQUAPAKS

A rapidly setting, specially developed cement (Aquacem) is mixed in a parallel mixer to form a slurry and pumped to the packhole. A parallel mixer and pump conveys a premixed water/bentonite and accelerator to the packhole where it is mixed with the grout before entering a brattice bag. The bentonite prevents the cement slurry from settling out and an acceptable pack can be prepared with a water: cement ratio of 2:1 by volume.

The advantages of this material are:

(a) Less cement is used.

(b) Economic mineral and/or coal is not lost to the pack.

(c) The equipment is simple.

The weight ratio of cement: Bentonite/accelerator: water for the Aquapak is 9:1:18.3.

2.2 - AQUALIGHT

As a light weight foamed cementitious cavity filler which is non-toxic and non-combustible. It requires a weight ratio of solid:water at 1:1, which when mixed, yields about 15 times its original volume. Every 100 Kg. of the material yields one cubic metre of foamed fill. After one hour of curing time, its compressive strength reaches to 0.05 MPa. Reducing amount of water yields stronger pack, but increases the coast of material, proportionately.

2.3 - ASTRAPAK AND TEKPAK

They are virtually the same type of low solids, water based packing materials. They are developed to reduce the material requirements and represent a 27% reduction in weight of material brought into the mine for each cubic metre of pack.

The method of placement into the packhole is identical to that of Aquapack.

The mix ratio is given below:

	Cement	Bentonite/Accelerator	Water
Astrapack	1	1	5.01
Tekpack	1	1	5.10

2.4 - ANHIDRITE PACKS (ANPAK)

Anhydrite (Ca SO_4) will in the presence of water, form dihydrite (Ca SO_4, $2H_2O$), a strong solid material. The transformation is far too lengthy for the needs of pack construction unless an accelerator is introduced. In anhydrite packs the accelerator used is a mixture of one part potassium sulphate (K_2SO_4) and 1.8 parts hydrated iron (II) sulphate ($FeSO_4$, $7H_2O$). The relationship of accelerator to anhydrite is 1% by weight. UK laboratory work and experience suggest the optimum ratio of water to anhydrite is 0.1, Whittaker et al [5].

The above paper details underground trials of the system at Easington and Parkside Collieries. For a single gate road two men were required. One at the bunker station and one at the packhole. Both trials were very successful.

2.5 - FLASHPAK

The beneficial effects of flyash addition in some concrete uses have been known for some time and cement/flyash mixture has been used for gateside packing in Japan and United Kingdom.

To make the pack, two grout mixing tanks and pumps with delivery tubes connected to an ejector nozzle at the packhole are needed. One mixer produces a 4:6 mix of cement:water, and the other a mixture of Flyash:accelerator:Bentonite:water of 10:1:1:4. The Flyash grout is stable for upto 4 days, and may be premixed and pumped from the surface. The cement grout has about 500 m pumping distance limit. The two grouts should be pumped at rates allowing a 3:1 Flyash grout:Cement grout mix at the mixing nozzle and sufficient velocity to allow the material to be sprayed to the back of the packhole. The mix gels almost immediately on mixing and can be built up like shotcrete in a packhole defined by steel sheeting. The pack becomes quite firm after 20 minutes. Packing rate can be 22 m^3/hour.

3 - FACTORS AFFECTING PACK BEHAVIOUR

The major factors affecting gate road packs are:

(a) Physical and mechanical properties of the pack material.

(b) Dimensions of the pack.

(c) Method of constructing the pack.

(d) Rate of the production.

(e) Rate of excavation closure and strata loading.

(f) Seismic effects.

In order to have the best effect in reducing the convergence, the pack should be installed immediately behind the faceline, before any major deformation, bed separation and/or fracture in the vicinity of the face is developed.

Whenever extraction of over 2 metres are concerned, the pack width should not be less than 3 metres. Pack widths of 3 to 10 metres have shown uniform loading pattern under normal strata pressures. Very narrow packs will fail quickly unless made of strong materials, in which case floor and/or roof penetration will occur which results in gate road deterioration.

In any case the strength of the pack material should match the strata loading and the strength of roof and floor in the vicinity of the pack.

The strata loading is affected greatly by the rate of face advancement in two ways:

(i) If the method of packing can not maintain the pack near to the face during rapid face movement, excessive convergence will occur in this area.

(ii) If the pack exhibits creep to any great extent, excessive deterioration may occur during long periods of face standstill or slow rate of advance.

4 - THE DEVELOPMENT OF PUMP PACKING SYSTEM

4.1 - PUMP PACK

A continous coal aggregate concrete packing system is known as "pump pack". In this chapter the parameters of strength, cost and mix constituents that are encountered in forming coal aggregate concretes are discussed. These results are then compared with other alternative methods of monolithic systems.

This system uses the run-of-mine coal , which is screened to extract the minus 19 mm material. This is stored at the bunker station to provide sufficient material for the formation of one pack. The bunker station consists of the bunker, extracted -19 mm coal, water and flow suspendent (bentonite) are mixed to make a pumpable slurry. The -19 mm coal material is fed into a paddle mixer and mixed with bentonite and water. The correct quantity of bentonite is controlled by a dispenser, the quantity depending upon the nature of the coal. The paddle mixer is hydraulically driven and feeds the material into the pack pump where it is mixed further. The pack pump which is used to pump the slurry mix is hydraulically operated and is powered by a 90 KW power pack.

The bentonite is stored at the pack pump and, as only small amounts are used, transportation is not a critical factor.

The slurry is pumped, with a 100 mm Victaulic pipeline to the face, where it is mixed with cement and accelerator slurry. Cement and accelerator are fed into the grout mixer via a dispenser and is thoroughly mixed with water before feeding to the grout pump. The mixture is pumped from the grout pump to the face with a 25 mm diameter double braided hose, where it is introduced to the pack with the slurry. A manual relief valve is installed in the grout line to discharge the grout, in the event of blockage in either the line or the pack. Both the mixer and pump are hydraulically operated and a 220 KW electric motor drives the hydraulic pump.

First a central mixing and pumping station are established to feed the face, and then the grout pumping station is situated in the roadway at close proximity to the face. The maximum pumping distance is approximately 2 km.

The pack is formed by shutter, constructed as the face advances. At the shutter, the slurry and grout are injected through a turbulent flow mixer into the shuttering to form the pack.

Here, the main constituents of the pump pack and their respective contribution to the cost and strength of the packs are discussed.

4.1.1 - Coal

The aggregate to be used for the pump pack mix should consist of run of mine coal (washplant rejects) sized to -19 mm, so that once it is mixed with water and bentonite, the slurry can be easily pumped through a 100 mm diameter pipe. Firstly the coal coming from the face should be screened,such that the -19 mm fraction is taken out and used as an aggregate. Secondly the -19 mm +6 mm fraction could be screened out. If this method is used then the -6 mm portion could be replaced by flyash or other suitable material. An aggregate consisting of evenly graded coarse and fine particles is a necessity for the mixing up of a high strength pack.

4.1.2 - Cement

The percentage of cement to be used in the pack is of great importance to both the cost of the operation and to the effectiveness of the pump pack.

There are a number of physical characteristics of the pack that are directly related to the cement content. These are as follows :

1: STRENGTH :

The compressive and shear strength of the pack will increase with increasing proportions of cement,

2: SHRINKAGE :

The higher the cement, the less is the shrinkage,

3: SETTING TIME :

The pump pack mix must be made up such that the frame can be removed after 18 hours, without the pack slumping. So the cement content of the mix must be high enough to meet this requirement.

4: WATER CONTENT :

In mixing up the cement before the grouting pump, water must be added at a minimum of 35 % by weight of cement.

4.1.3 - Water

The final water content of the pump pack will depend largely upon the amount of water that is needed to form a pumpable slurry when mixed with the aggregate and bentonite.

4.1.4 - Bentonite

Bentonite is a ground clay that is added to the coal and water slurry to assist the flow of the slurry through the pipeline and reduce the tendency for the water to separate out of the mix and concentrate at the high points of the pipeline. Both the water content and compressive strength of the concrete mixes are affected by the presence of bentonite. Bentonite on contact with water swells up to eighteen times its original volume by the absorption of water. Therefore the addition is desirable for the hydration of cement, and that more water than usual must be added to make the mixture workable.

The major disadvantages of pump pack system are :

1 : Increased floor lift,

2 : Failure to totally eliminate spontaneous combustion,

3 : High capital cost,

4 : High methane concentrations at the tailgate face-end necessitating methane drainage of the waste through the pumped packs.

4.2 - THE THYSSEN SYSTEM

This is one of the original and most widely used systems (Figure 1). An outbye mixing station is established at which 19 mm run of mine coal is mixed with bentonite and water to form a thick slurry.

The production of coal and the needs of pack construction do not always coincide so it is necessary to provide some form of bunker to store the run of mine coal.

The slurry is pumped along 100 mm pipe ranges to the packhole areas, a valve allowing selection of either gate.

In the return gate, so as to reduce dust pollution on the unit , a cement grout mixer is sited. This mixes a quick setting cement (Packbind) with water and pumps it to either gate along 25 mm or 50 mm hose where it is introduced into the aggregate pipeline near the delivery nozzle.

Hand set shuttering was originally used but this was time consuming and often the packhole was not filled to the roof due to leakages and gradients. Purpose built shuttering was introduced which was advanced by the packhole and buttress chocks. Adequate containment of the pack mixture was still a problem and there was a tendancy for the packs to be dragged forward if not properly set. Ideally, 2 hours should elapse before the shuttering is removed. Self-advancing hydraulic shuttering also tended

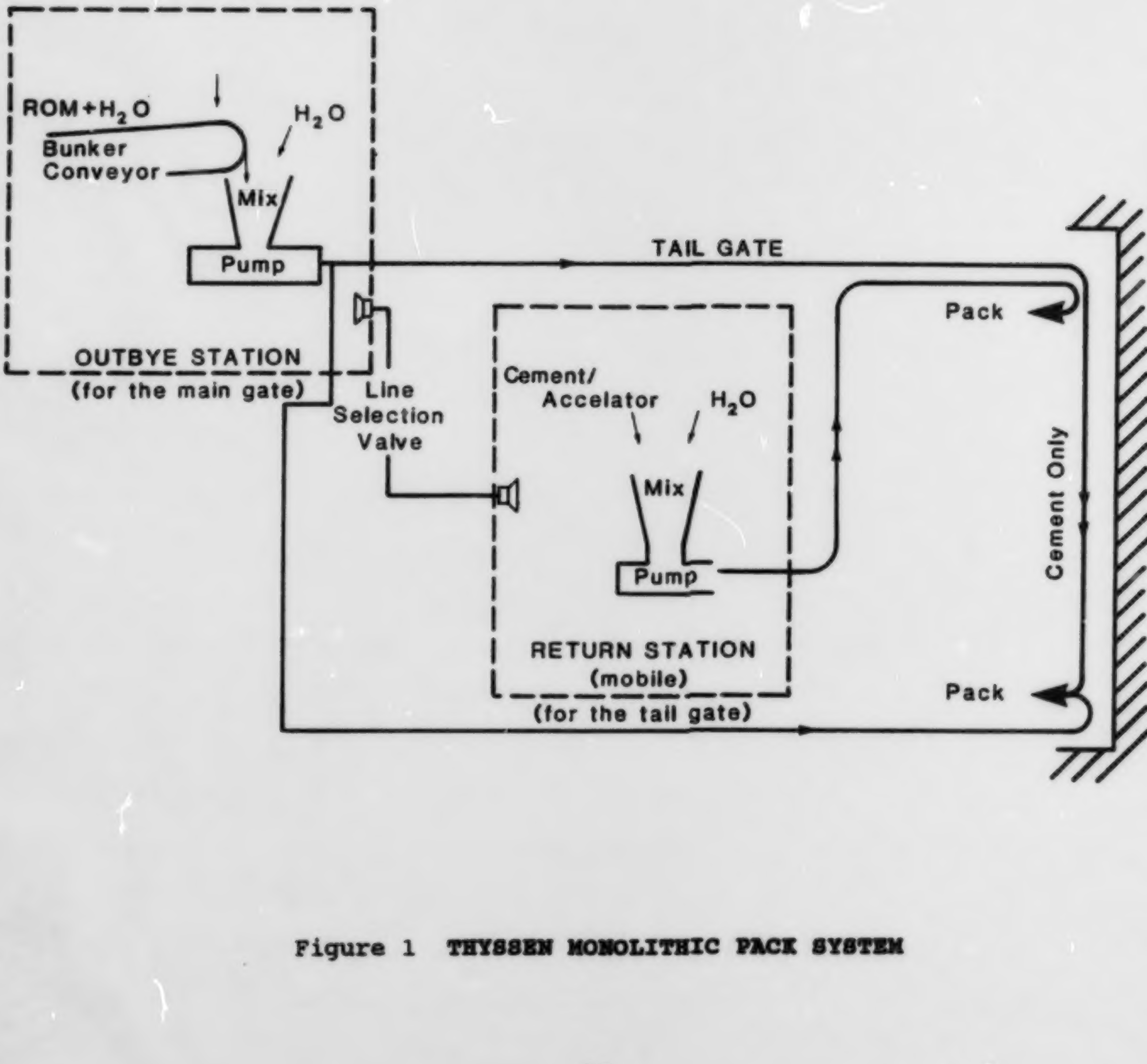

ROM+H_2O
H_2O
Bunker
Conveyor
Mix
Pump
OUTBYE STATION
(for the main gate)
TAIL GATE
Pack
Line
Selection
Valve
Cement/
Accelator
H_2O
Mix
Pump
RETURN STATION
(mobile)
(for the tail gate)
Cement Only
FACE
Pack

Figure 1 **THYSSEN MONOLITHIC PACK SYSTEM**

to suffer these disadvantages and, additionally, occupied too much space. The use of brattice cloth bags supported by weldmesh has solved these problems. Careful utilization can allow setting of packs to the roof even when there is a gradient within the packhole.

As with all such systems it is necessary to flush out the grout line after each pack operation. This should not be the case with the aggregate line but blockages did occur with the Thyssen System. These resulted from poor mixing, incorrect ratios of the various materials or the escape of water (bleeding) causing segregation of the water and solid constituents.

Complaints were received that the pack material was not always fully mixed which can lead to the collapse of the pack. This seems to arise from an excess of bentonite causing the aggregate slurry to be so viscous that a proper grout/slurry mix was unobtainable.

Facilitating pumping by increasing the water content of the slurry can lead to the cement grout flowing too readily giving grout free aggregate in parts of the pack.

An improved mixing nozzle has been designed and tested for future applications.

A further problem with the system is that Packbind is a skin irritant.

4.3 - THE WARBRET SYSTEM

The system was developed within the NCB (the name is derived from Warwickshire and Bretby, the Mines Research and Development Establishment). The system is a modified Thyssen System.

More recently, Mindev (Figure 2) and MRDE systems have undergone successful surface trials.

4.4 - PUMP PACKING PERFORMANCE

In general, pump packing has been greeted enthusiastically by those who have applied it. When reviewing pump packing at Newdigate and Daw Mill Collieries, McCarthy and Robinson in U.K. (2) were more guarded in their optimism.

The new system designed to overcome the shortcomings of the old one is shown in Figure 3.

The likelihood of pipe blockages due to bleeding and segregation are diminished by pre-mixing the bentonite suspension so that gelling is complete before it is added to the aggregate. In the Thyssen system gelling of the suspension takes place in the pipeline where it is inhibited by the aggregate.

A special cement accelerator TEA51 developed at MRDE, Bretby is added to the ROM coal. The TEA51 neither inhibits nor is inhibited by the bentonite provided that the bentonite is pregelled. With commercial cement accelerators the pumpability of the bentonite or the accelerated setting times would be affected.

Ordinary Portland Cement (OPC) can be used instead of Packbind. Pre-gelling of the bentonite enables the correct amount of water to be used so excess cement consumption

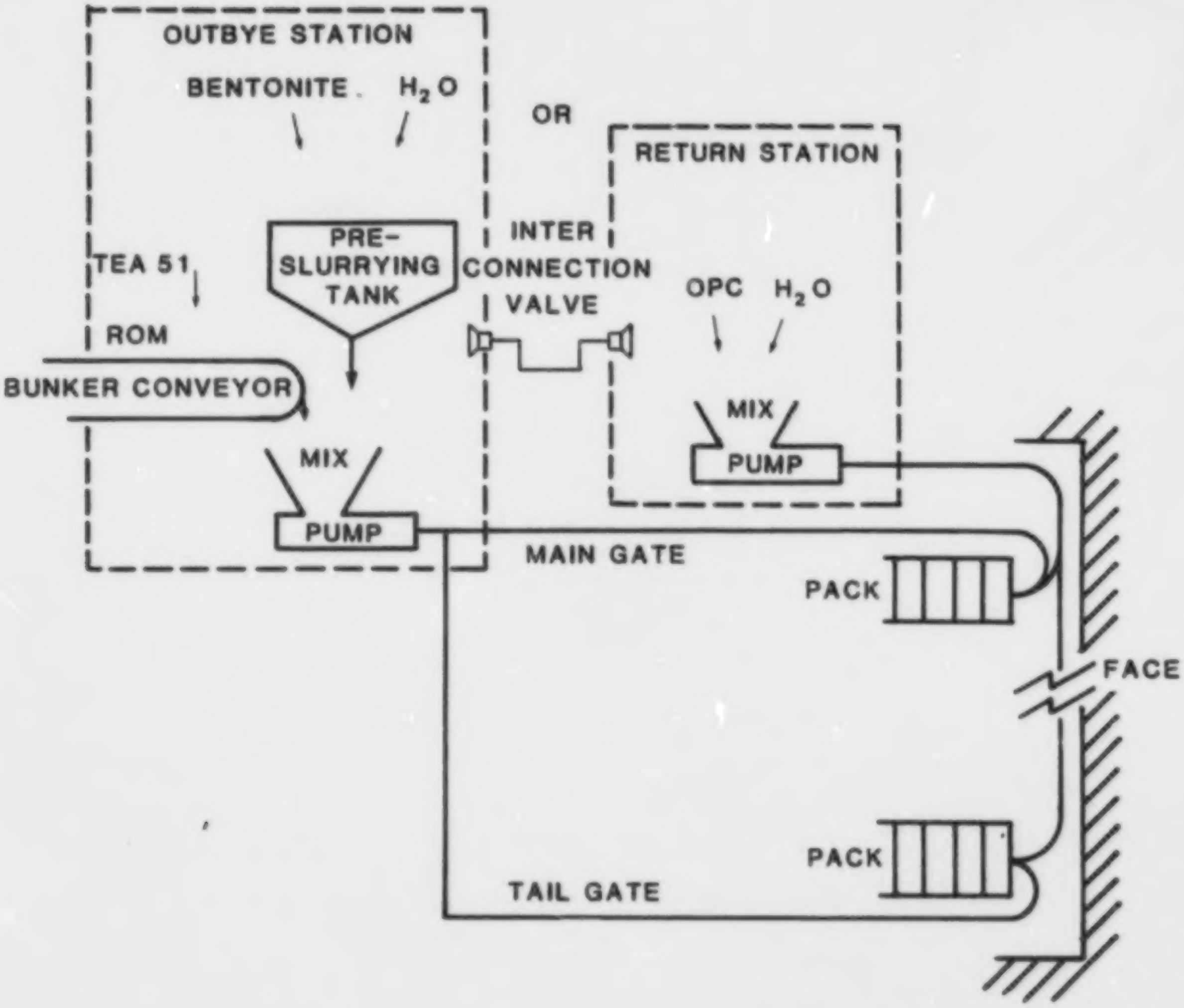
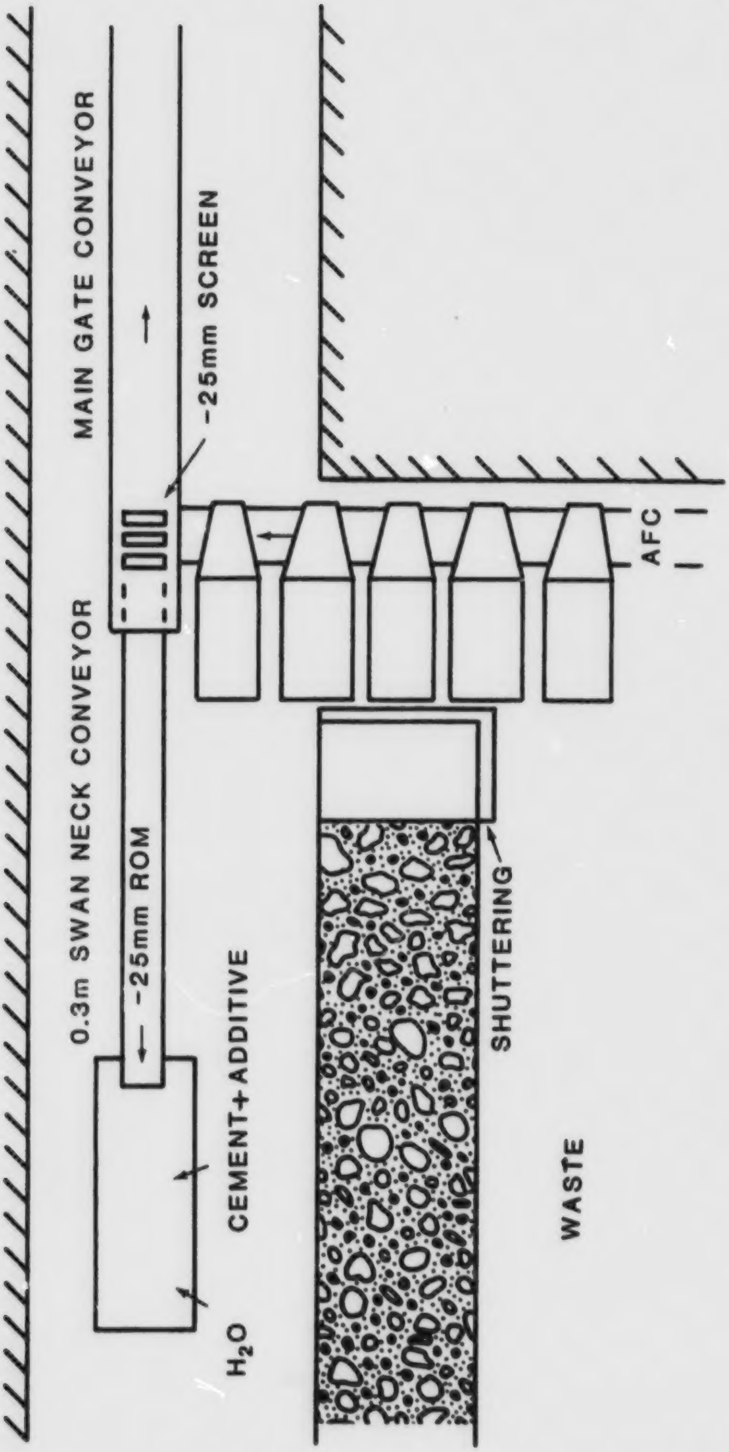

OUTBYE STATION
BENTONITE
H_2O
OR
RETURN STATION
INTER CONNECTION VALVE
PRE-SLURRYING TANK
TEA 51
OPC
H_2O
ROM
BUNKER CONVEYOR
MIX
MIX
PUMP
PUMP
MAIN GATE
PACK
FACE
PACK
TAIL GATE

Figure 2 **WARBET MONOLITHIC PACK SYSTEM**


MAIN GATE CONVEYOR
-25mm SCREEN
AFC
0.3m SWAN NECK CONVEYOR
-25mm ROM
CEMENT+ADDITIVE
H_2O
SHUTTERING
WASTE

(sometimes 33% of consumption) is eliminated. This gives rise to a cost saving and a reduced load on the materials handling system.

4.5 - SHORT RANGE SYSTEMS

These systems are not common. They involve the batch mixing of all the pack constituents in the gate close to the packhole. The elimination of long range pumping means that bentonite is unnecessary but the system uses space in an already congested area. The equipment advances with the face.

An early system installed at Kinneil/Valleyfield Colliery (U.K.) in 1973 was unsuccessful due to the pack being too brittle.

The greater load bearing capacity of the pump pack reduces arch distortion but transfers more load to the floor causing floor failure and subsequent heave. However, dinting is preferable to back-ripping and the floor could possibly be controlled by dowelling. Pack closure is down to 15-20%.

Although spontaneous combustion has not been totally eliminated there has been a vast reduction in the number of heatings on faces using pump packing.

The high capital cost is due in the main, to the coal pumping station and bunker and is unavoidable with present systems. The method of surface pumping of tailings would have a number of advantages. An entirely worthless aggregate would be used. The surface installation would be permanent and service far more faces. The packing operation would be less reliant on face production allowing more flexibility.

A logical additional step would be to pneumatically pump the cement from the surface. This would reduce the load on the supplies system, possibly eliminate the need for a grout pump and, coupled with the use of tailings, reduce manpower at multiface collieries.

Until surface pumping of the cement is practical improved handling could be provided by introducing a shredding device so that the cement bags could be loaded unopened. This would increase the rate of feed, reduce dust and dispose of the bags safely.

The necessity for methane drainage arises from the more efficient ventilation of the face-ends. This is a result of the impermeability of the packs preventing short-circuiting of the air across the waste.

5 - LABORATORY INVESTIGATIONS ON THE MONOLITHIC PACKS

5.1 - TESTING TECHNIQUES

A compression testing machine with capacity of 20 tons and capability of applying constant rate of displacement was employed for all the testing programmes.

The uniaxial compressive strength tests were conducted on 50 mm diameter x 100 mm length cylinders at a displacement rate of 1 mm per minute. A minimum of five uniaxial or three triaxial tests were carried out for each parameter set.

The platens used for these tests were of the same diameter and height as the sample. This produces a better stress distribution than that associated with many platen configurations.

A latex rubber sheath was used during these tests to protect the sample for the air.

The test procedures accord with the standards of the I.S.R.M. [1].

5.2 - AQUAPAK TESTS

5.2.1 - Preparation of Samples

Aquapak was the first cementitious pack material made available for testing. A period of familiarization was therefore necessary in order to ensure that the prepared samples could be confidently regarded as representing the material. Samples were prepared and failed on the compression testing machine. When the results repeatedly agreed to within 10% for the group samples each within the range 1 day to 1 month, the sample preparation was considered to be of a suitably competent and consistent nature.

The standard or correct mix is suggested to be: 1 volume of Aquacem grout comprising 0.8 parts water and 1 part Aquacem by weight. 1 volume of Aquabent slurry comprising a 10% concentration of Aquabent.

The grout and slurry stood for 5 minutes prior to being mixed together. Vigorous hand mixing was employed. This caused no separation of solids from water. The Aquapak was then poured into plastic (50 mm diameter) moulds. After 1-2 hours it was possible to remove the samples from their moulds.

The mixing water temperature was 25°C and the samples were cured in plastic bags at 25°C.

When a sample is said to have an "excess" of a constituent this excess is by weight.

5.2.2 - AQUAPAK DENSITY

Aquapak prepared with the correct proportions has a density of 1.375 tonnes m^{-3} upon initial set. With a 50% excess of Aquacem this figure rises to 1.475 tonnes m^{-3}. A 50% excess of Aquabent produces a mix with a density of 1.373 tonnes m^{-3} upon initial set. A similar excess of water will cause this value to fall to 1.260 tonnes m^{-3}.

After a month the value for the correct mix falls to 1.368 tonnes m^{-3} when cured in a plastic bag at 25°C. The density values of various Aquapak mixed at different temperature after 30 days of curing are given in Table 1.

Table 1 - Aquapak Density After 30 Days Of Curing

Mixing/Curing Conditions	Density, tonnes.m^{-3}
50% excess Aquacem	1.475
50% excess Aquabent	1.369
50% excess water	1.243
Air cured	0.692
Cured 1 month bag/1 month air	0.609
50% Air cured	0.790
Fraction air cured	1.299
Cured at 9°C	1.387
Cured at 60°C	0.570
Mixing water temp. 5°C	1.291
Mixing water temp. 40°C	1.278

Clearly, exposure to the air or high temperatures causes excessive water loss.

5.2.3 - UNCONFINED COMPRESSIVE TESTS

(a) For the standard and Excess Mixes

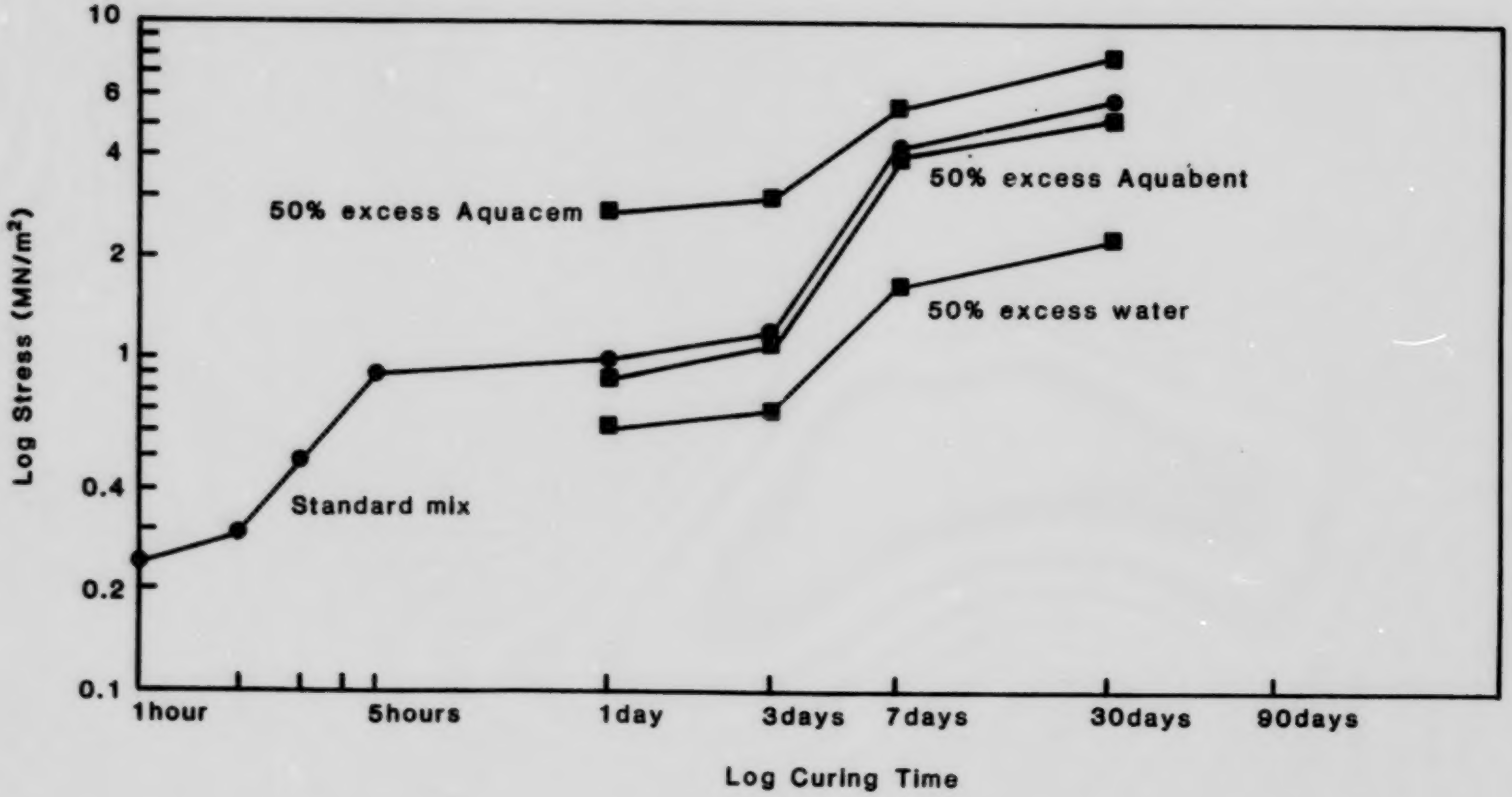
Figure 4 shows a log plot of strength against curing time for the stamdard mix and for 50% excesses of each constituent. Typical stress/strain curves are given in Figure 5 - 8.

The standard mix exhibits good early strength followed by rapid strength development from 2 to 5 hours. The strength does not rise significantly again until 3 days when, up to 7 days, there is a further rapid rise.

The form of the curve is a function of the very complicated cement chemistry of this mixture.

Up to three days the stress/strain curves (Figure 12) display a plastic form; steady yield under stress to a low peak strength followed by only a small loss of strength. At four days and after the rate of increase is more rapid and leads to a more significant peak strength. Once the peak strength is reached there is a rapid fall off in strength as the material breaks up. Such behaviour is characteristic of a brittle material. This transformation from plastic to brittle behaviour is associated with a lightening of the material's grey colour.

The curves for excess Aquacem and excess water (Figure 4), have a similar trend to the correct mix. It must be borne in mind that, whilst proving a marked success in mining, Aquapak is


10
6
4
2
1
0.4
0.2
0.1
Log Stress (MN/m²)
1hour
5hours
1day
3days
7days
30days
90days
Log Curing Time
50% excess Aquacem
50% excess Aquabent
50% excess water
Standard mix

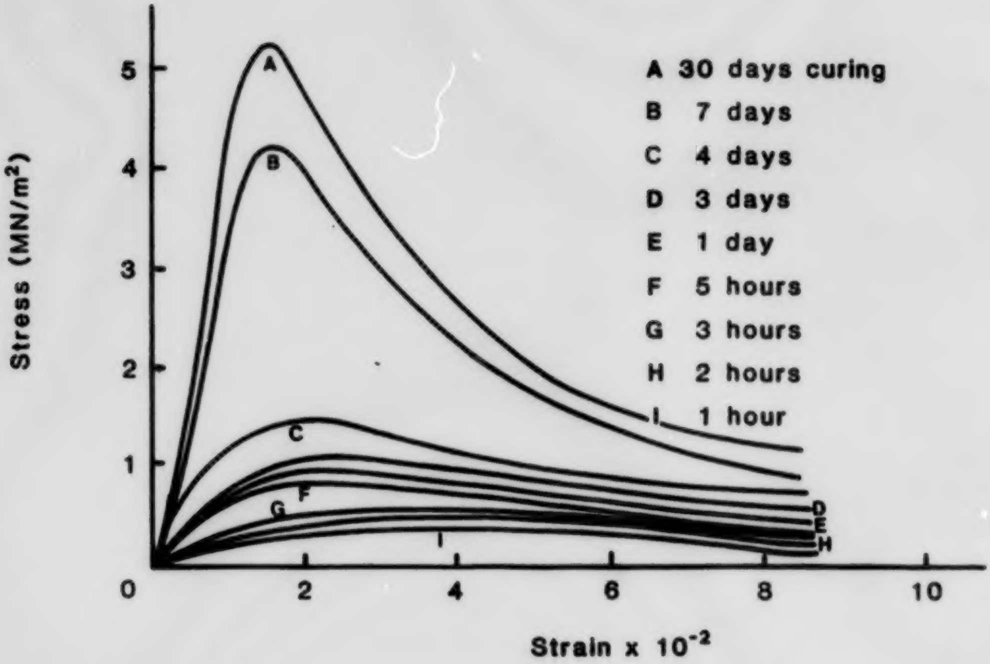


Figure 5 **STRESS/STRAIN CURVES FOR STANDARD MIX**

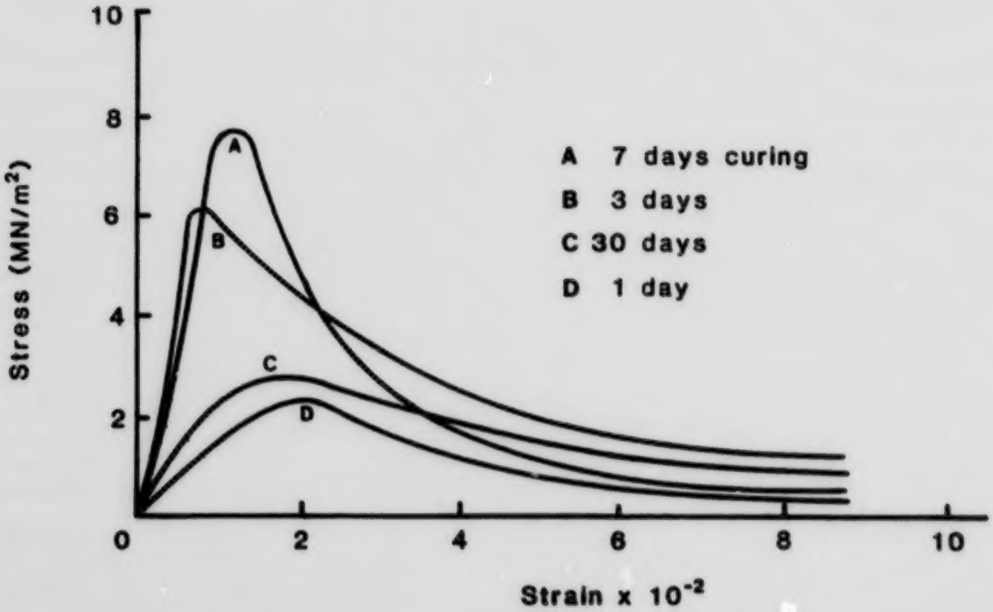


Figure 6 **STRESS/STRAIN CURVES FOR EXCESS AQUACEM**

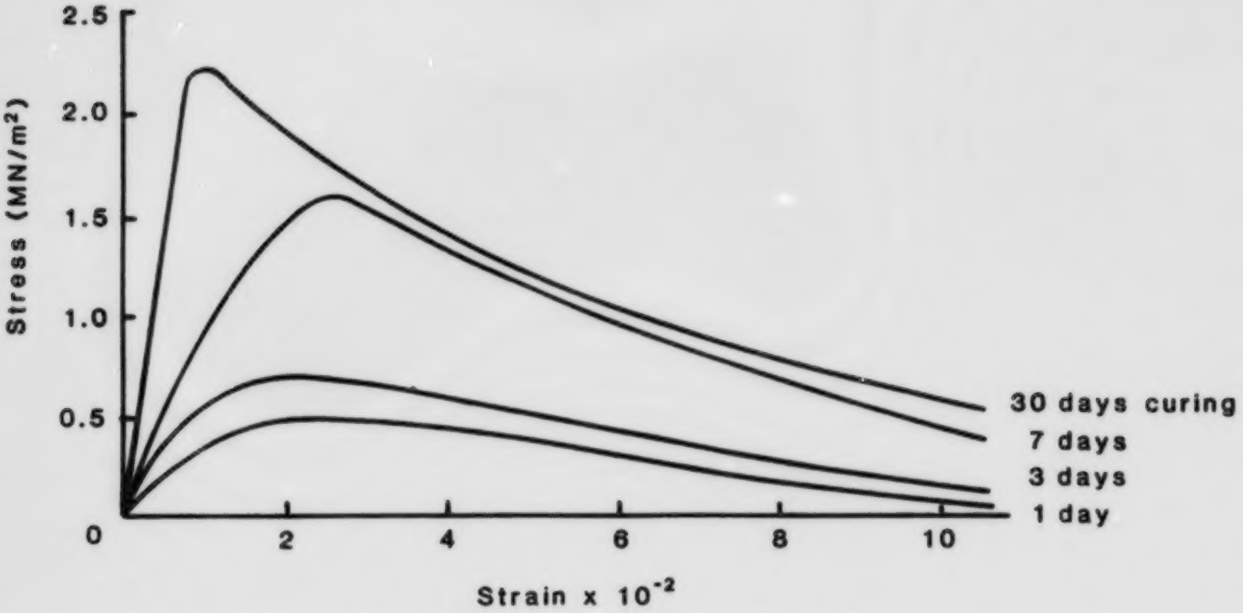


Figure 7 **STRESS/STRAIN CURVES FOR EXCESS WATER**

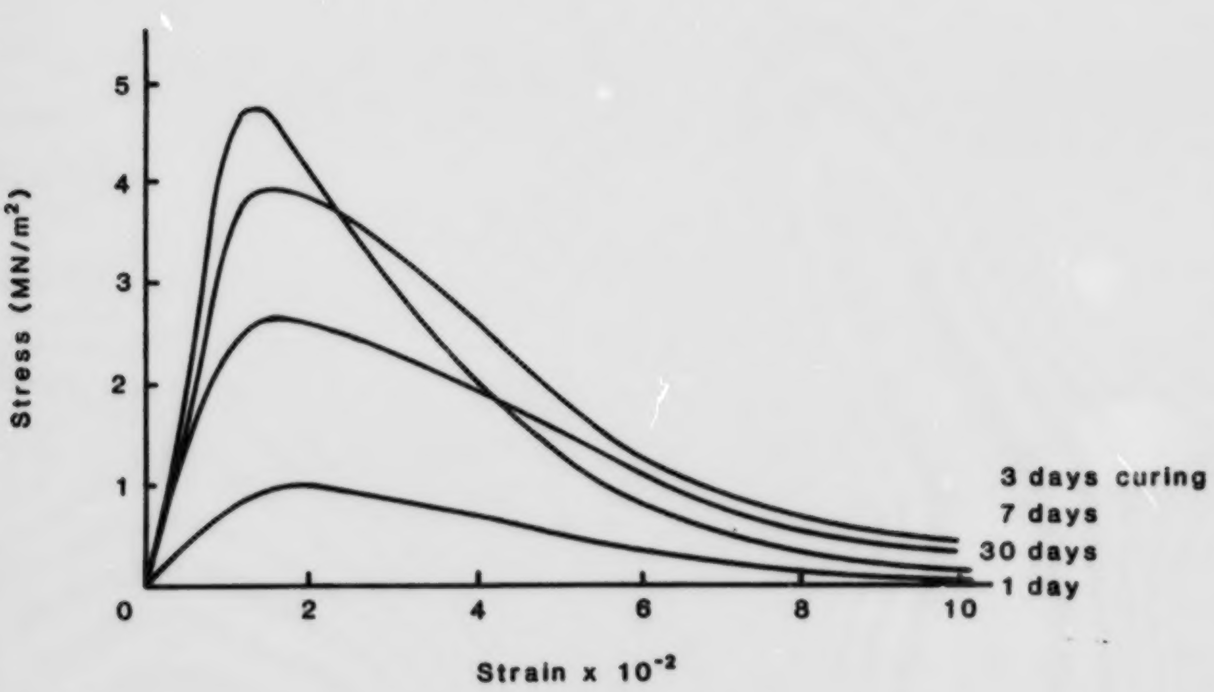


Figure 8 **STRESS/STRAIN CURVES FOR EXCESS AQUABENT**

a very weak cement. This is a direct result of the large proportion of water. Therefore, increasing the amount of water will further weaken the cement, conversely increasing the cementitious content will strengthen the cement. Up to 1 week the excess water samples shed water under compression.

An unnecessarily strong cement adds needlessly to material costs. A too weak cement could prove very much more expensive, as a result, roof control is lost. In the crudest Aquapak systems, in which bags are manually opened and loaded into gate hoppers, the latter is very much more likely. Underground storage silos or, preferably, pneumatic transport would allow far greater control over the quantities of each constituent used. It is clear from Figure 11 that the packing systems have an in-built flexibility that might be useful where conditions are heavier than usual.

The excess Aquabent curve (Figure 4) displays a more uniform increase in strength between 1 day and 7 days. The strengths of Aquapak with a 50% excess of Aquabent are, however, largely the same as those for the respective correct mix. This indicates that the correct mix itself has an excess of Aquabent. The further excess cannot accelerate the hydration of the cement and is not large enough to inhibit the strength development.

(b) Change of Mixing and Curing Parameters

A number of parameters were altered in order to investigate the effect of such an action on a correct mix.

Using cold water ($10^{o}C$) to prepare the samples does not significantly effect the long term strength. The thirty day strength being 4.37 MPa. Time to initial set was 15 mins instead of the usual 5-7 mins, this, the lower density (Table 1) and the reduced strength indicate that low water temperatures do inhibit hydration of the cement.

Warm water does have a markedly detrimental effect. The average strength was only 2.47 MPa. Of great importance is that the use of warm water produced a number of samples that conformed to the definition of a "bad mix". These samples behaved plastically with a peak strength of 0.91 MPa.

One batch of samples was left to stand without mixing. This batch produced samples of varying strength (2.90-4.49 MPa) and varying density (1.323-1.418 tonnes m^{-3}). Generally, samples with the greater density exhibited the higher strength. Clearly separation of the constituents takes place producing samples with an excess of one or other constituent. An excess of the higher density cement producing samples of higher strength.

Curing at low temperatures ($10^{o}C$) has as little affect as mixing at low temperatures. Samples so cured had a thirty day strength of 4.39 MPa. Curing at $60^{o}C$ temperature, however, dehydrates the samples resulting in low strengths, 0.81 MPa.

The effects of air on a correct mix were investigated. A number of samples were left to cure completely exposed to the atmosphere. A further set were left to cure with 50% of their surface area exposed and a further set had just a tiny fraction of their surface area exposed. This was carried out with the aid of saran wrap. Another set of samples was cured in a plastic bag for 1 month and then cured for an additional month exposed to air. The curing temperature was maintained at 25°C.

As mentioned earlier, exposure to air causes a change of colour and a reduction of the density. It also greatly reduces the strength to 2.47 MPa.

Allowing only 50% exposure to air remarkably reduces the strength yet further to 1.60 MPa. The unexposed half remained unaffected by the test. The uneven dehydration of the sample produced a large unconformity that resulted in the seemingly anomolous low strength.

The effect of only fractional exposure is insignificant. The strength of 5.04 MPa being only marginally below the 5.17 MPa of a standard mix.

The density of the samples cured in a plastic bag for one month was lower than that for the fully exposed thirty days samples and the strength was also lower at 2.14 MPa.

To appreciate the reason for this it must be understood that set cement contains two types of water [10]. These are evaporable water and non-evaporable water. The former is held in pores within the structure or gel of the cement whereas the latter is water of hydration of set cement compounds. In reality the division is not quite so clear but such a simplified picture is adequate for present purposes.

The samples initially set in a plastic bag will develop full strength with a full compliment of water. When dried out the structural pores, formed about the water, and the gel pores will have lost their water, so weakening the cement.

The samples that were exposed to the atmosphere from mixing will have lost water by evaporation immediately. This will have reduced the water available for hydration so inhibiting strength development. However, by way of compensation the water/cement ratio of subsequent hydration will have been reduced, and the number and size of structural pores will also have been reduced.

The results illustrate that the drying out process is a slow one and that small tears are unlikely to affect the strength of an Aquapak. Drying out on a large scale would be detrimental. The differential drying associated with larger exposed volumes of material produces differential contraction of the cement, which can induce failure, Lea [2]. The probability of such an event underground does not seem very likely. That the bag does more than just retain the Aquapak when liquid is, however, well illustrated.

5.3 - ASTRAPAK AND TEKPAK TESTS

5.3.1 - TEKPAK TESTS

There is much similarity between the application of Aquapak and Tekpak. The cement and bentonite products are marketed as

Tekcem and Tekbent, respectively. They are mixed in the ratio of 1 to 1 by weight. The proportion of water in the pack is 91% by volume, Newson [3].

The initial density is 1.281 tonnes m^{-3}, falling to 1.221 tonnes m^{-3} after 1 week.

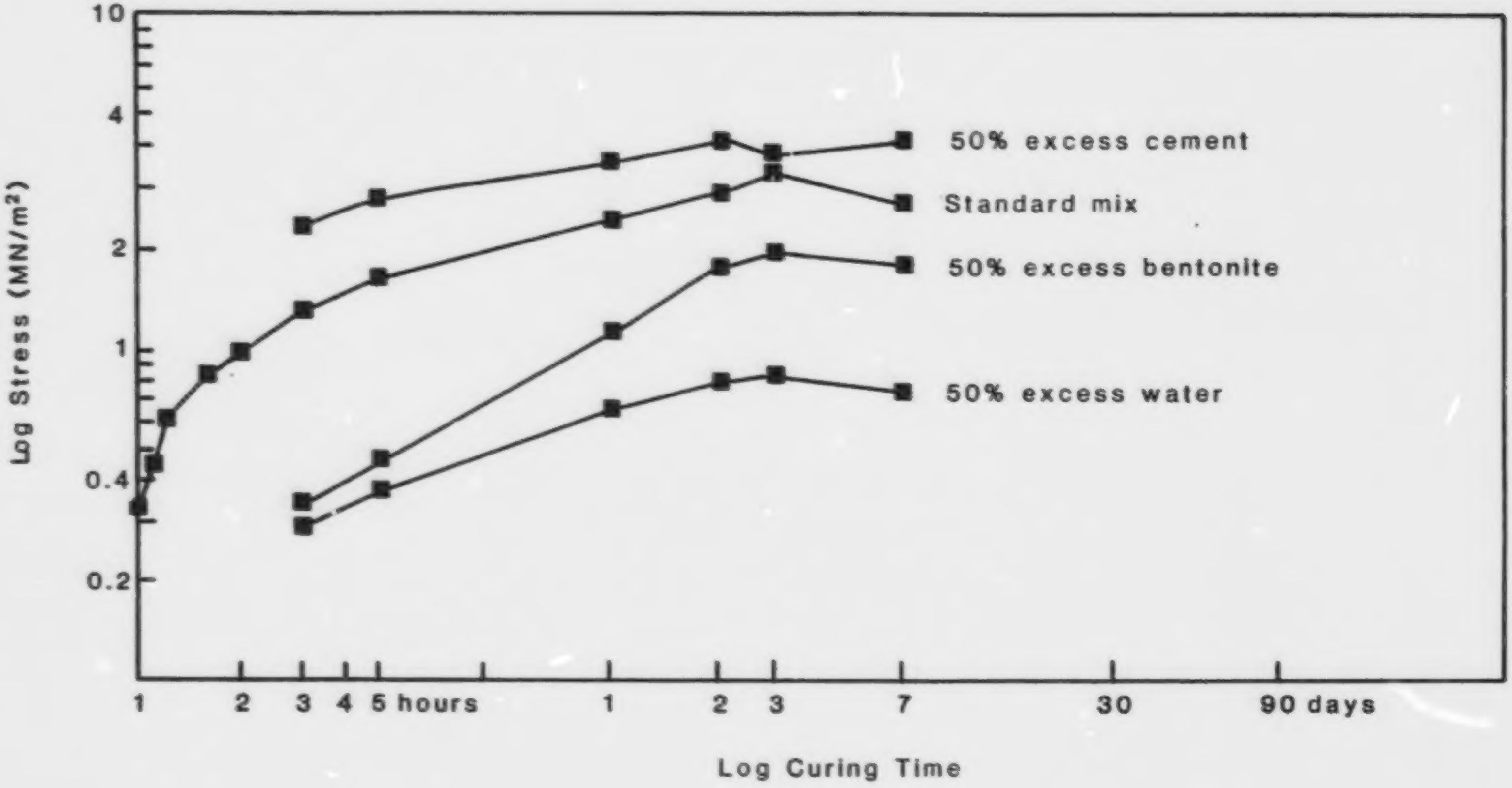
5.3.1.1 - Sample Preparation

The two solid constituents were prepared separately with a water/solids ratio of 2.5 to 1 by weight. After standing for ten minutes the grout and slurry were mixed together in a 1 to 1 volume ratio. Tekpak relies on its own generated heat to ensure correct curing. The samples were kept close together and insulated by a number of cloths so that curing was not adversely affected.

5.3.1.2 - Unconfined Compressive Tests

Figure 9 gives a log plot of strength against curing time for correct and excess mixes. Typical stress/strain curves are given in Figures 10-14.

The correct mix exhibits very good early strength development up to 5 hours. The strength then develops steadily to a peak strength of 3.37 MPa at 3 days. After this the strength tails off to 2.73 MPa after 7 days. The curve is characteristic of some high alumina cements. The transition from plastic to brittle behaviour appears to occur between 5 hours and 1 day (Figure 10).



Log Stress (MN/m²)
10
4
2
1
0.4
0.2
1
2
3
4
5 hours
1
2
3
7
30
90 days
Log Curing Time
50% excess cement
Standard mix
50% excess bentonite
50% excess water

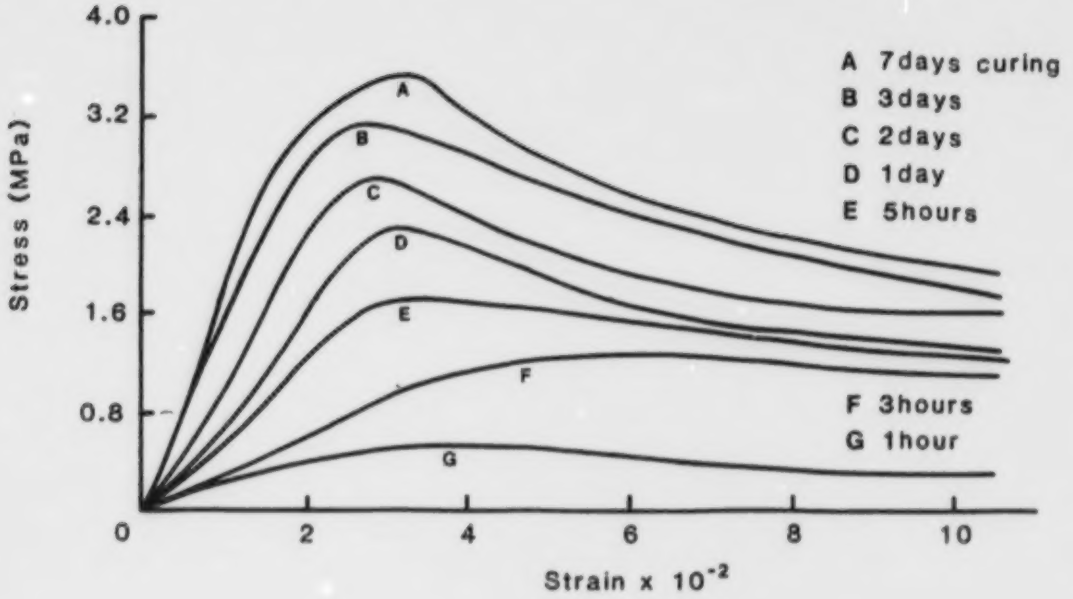


Figure 10 **STRESS/STRAIN CURVES FOR STANDARD MIX**

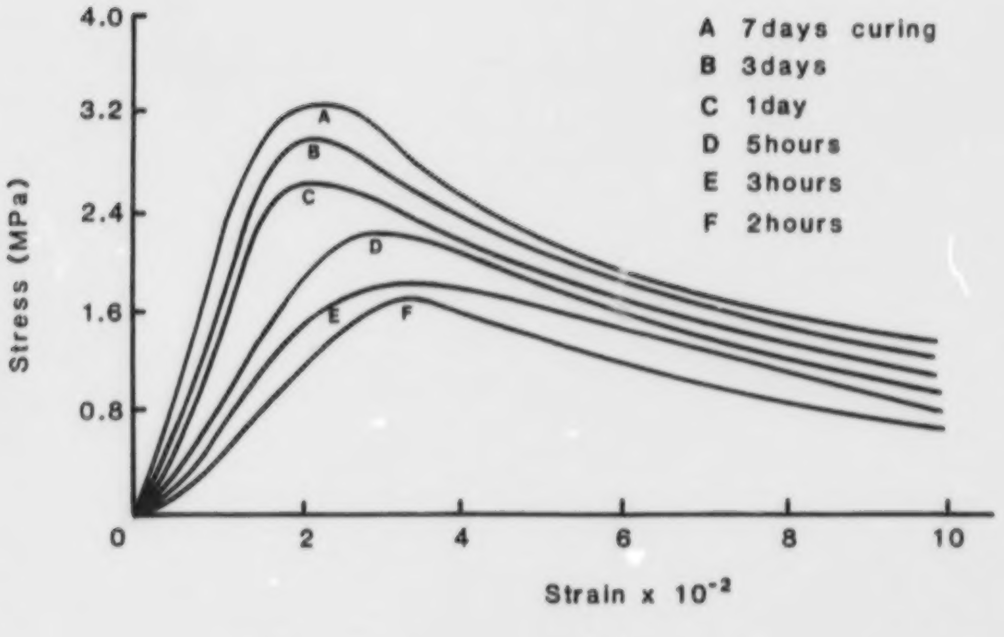


Figure 11 **STRESS/STRAIN CURVES FOR EXCESS TEKCEM**

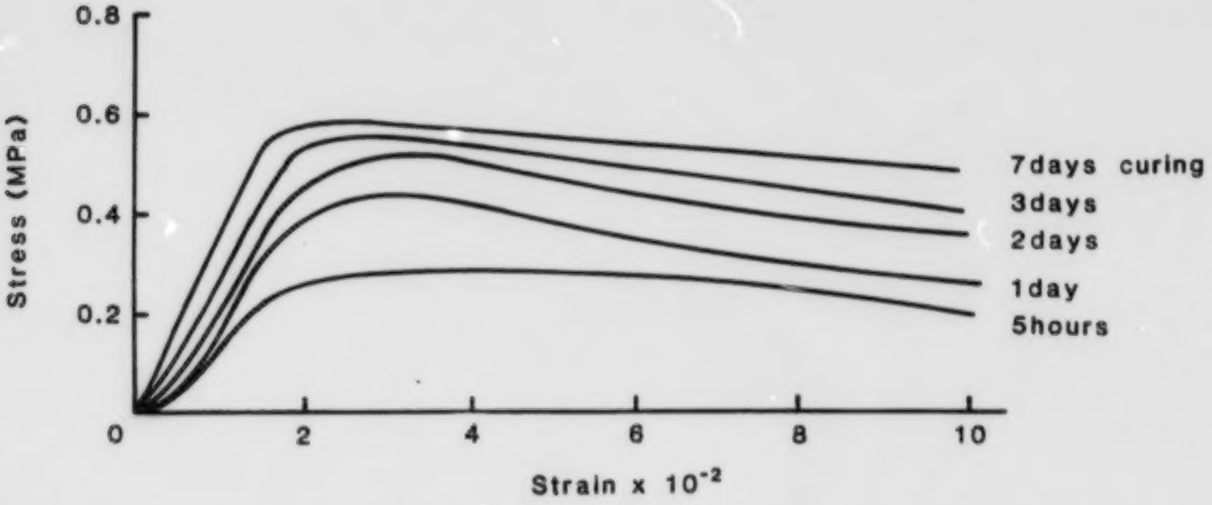


Figure 12 **STRESS/STRAIN CURVES FOR EXCESS WATER**

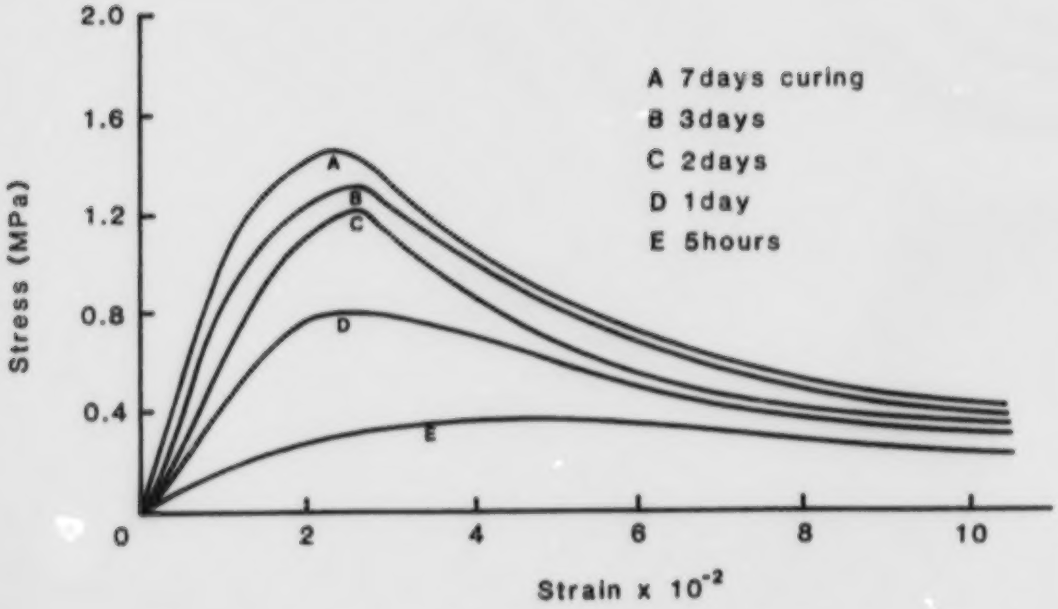
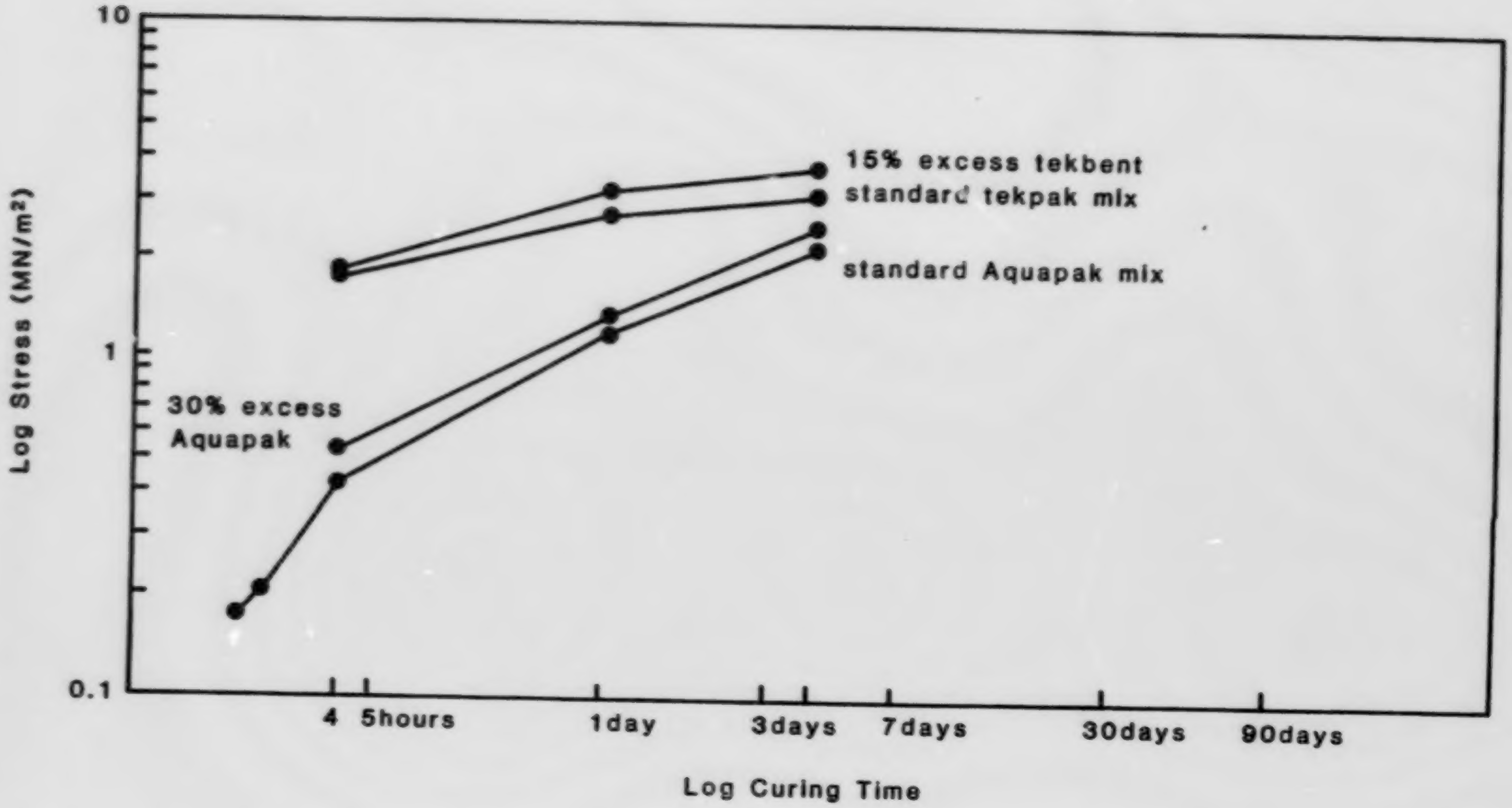


Figure 13 **STRESS/STRAIN CURVES FOR EXCESS WATER**


10
1
0.1
Log Stress (MN/m²)
15% excess tekbent
standard tekpak mix
standard Aquapak mix
30% excess
Aquapak
4
5hours
1day
3days
7days
30days
90days
Log Curing Time

The other three curves on Figure 21 are similar to that of the correct mix. The Aquapak curve rises more steeply to 2 days. The Aquacem curve peaks after 2 days but the tail off is less apparent.

The 50% excess Tekbent has a marked affect on the strength of the cement. It must be noted that 50% Tekbent represents a much larger proportion of the solids than does 50% Aquabent. The former means that the final cement will contain 50% more Takbent than Tekcem by weight. By contrast 50% more Aquabent means that the Aquacem/Aquabent weight ratio is reduced from 9 to 1 to 6 to 1, still a substantial ratio in favour of Aquacem.

The result of this excess bulk of Aquabent is to reduce the strength but to greatly increase the plastic behaviour. The 5 hour samples closed 10 mm (10%) without any cracks evident. The samples "barrelling" in a classic plastic failure fashion. The samples were then removed from the press and allowed to cure for a further 19 hours before being retested. The samples then had a mean strength of 1.15 MPa compared to 1.10 MPa for the 1 day tests.

The 50% excess water is detrimental to the all round performance of the cement. The 50% excess Tekcem produces impressive early strengths but the later strengths are low in the light of those achieved by a 50% excess of Aquacem.

5.3.2 - ASTRAPACK TEST

An extensive laboratory testing programme was conducted on the physical and mechanical properties of the Astrapak material in order to investigate:

a. Effect of curing time on the density
b. Effect of curing time on the P-wave velocity through the specimen
c. Effect of curing time on the modulus of elasticity
d. Effect of curing time on the unconfined compressive strength
e. Effect of water and curing temperature on the above noted parameter
f. Effect of sample size on the internal increase of sample temperature due to chemical reaction during the curing period
g. Effect of excess water and cement on the mechanical properties of Astrapak
h. The mechanical behaviour of the Astrapak under very slow strain rate (long term compressive strength test).

5.3.2.1 - SAMPLE PREPARATION

(a) Sample Preparation for Standard Mix

The correct standard mix for Astrapak recommended by the manufacturer is:

Astrabent:Astracem:Water = 1:1:5.01

This mixture was prepared by mixing 2.5 parts of water with one part of Astrabent in a mixer for 2 minutes to form a creamy coloured slurry. Similarly 2.51 parts of water was mixed with one part of Astracem for 2 minutes to form a dark gray slurry.

The Astrabent slurry was mixed manually for a few seconds prior to the Astracem slurry being poured onto it. The final mixture was stirred manually for 1 minute prior to being poured into cylinderical specimen moulds with internal diameter of 50 mm and height of 130 mm. The samples were left in the moulds for 45 minutes before it was taken out and the ends were cut and ground. The final length of the specimens were 100 mm. Each individual samples were wrapped in saran wrap. The standard mix specimens were prepared with water which was at the temperature of 12°C and 25°C.

(b) Preparation of Specimens with 50% Excess Water

These samples were prepared in similar manner for the standard mix discussed earlier. However 50% more water was introduced to each slurry. The water and curing temperatures for these samples were kept at 25°C.

(c) Preparation of Specimens with 50% Excess Astracem

Similar technique as that employed for preparation of the standard mix specimen were adopted for these batch of sample. However 50% more Astracem was added to the slurry.

(d) Preparation of Specimen for Long Term Tests

Standard mix was used for these specimen. The water and curing temperature was kept at 25°C.

(e) Preparation of Specimen for Temperature Measurement During Curing

Standard mix slurry described earlier was employed for these tests. The slurry, however, was poured into cubic moulds of 5, 10, 15 and 25 cm. This gave specimen surface areas of 150, 600, 1350 and 3750 cm^2 and volume of 125, 1000, 3375 and 15626 cm^3 respectively.

5.3.2.2 - TESTING PROCEDURE

(a) Density Measurements

In order to determine the density of each sample the weight and volume of each particular sample was measured after 1, 1.5, 2, 2.5, 3, 6, 24, 48, 72, 168 and 720 hours of curing time. The results for each batch of samples regarding any particular curing time is averaged and presented in Table 2 and Figure 15.

(b) Determination of Unconfined Compressive Strenth

The unconfined compressive strength tests were conducted at a slow rate of deformation of 0.017 mm/sec. The deformation was measured by an L.V.D.T. placed between the platens. The load and deformation measurements were directly recorded by a computer control data acquisition system. The summary of the averaged results are presented in Tables 3 and shown in Figures 16-22.

Table 2 CHANGE IN THE DENSITY OF ASTRAPAK DURING CURING (AVERAGE OF 5 TESTS)

Type of Sample			Density ( ), in gr/cm³									
Type of Mixture in Sample	Water Temp. (°C)	Curing Temp. (°C)	Curing Time (hours)									
			1	1.5	2	2.5	3	6	24	48	72	160
Standard	12	10	1.48	1.47	1.47	1.46	1.46	1.44	1.44	1.44	1.44	1.44
Standard	12	25	1.57	1.56	1.54	1.53	1.52	1.50	1.36	1.22	1.15	1.07
Standard	25	10	1.58	1.56	1.56	1.53	1.51	1.46	1.46	1.46	1.46	1.46
Standard	25	25	1.48	1.47	1.46	1.45	1.45	1.42	1.31	1.24	1.21	1.19
50% Excess Water	25	25	1.52	1.50	1.49	1.48	1.48	1.44	1.32	1.16	1.13	1.01
50% Excess Astracen	25	25	1.61	1.59	1.57	1.56	1.55	1.52	1.40	1.26	1.20	1.15

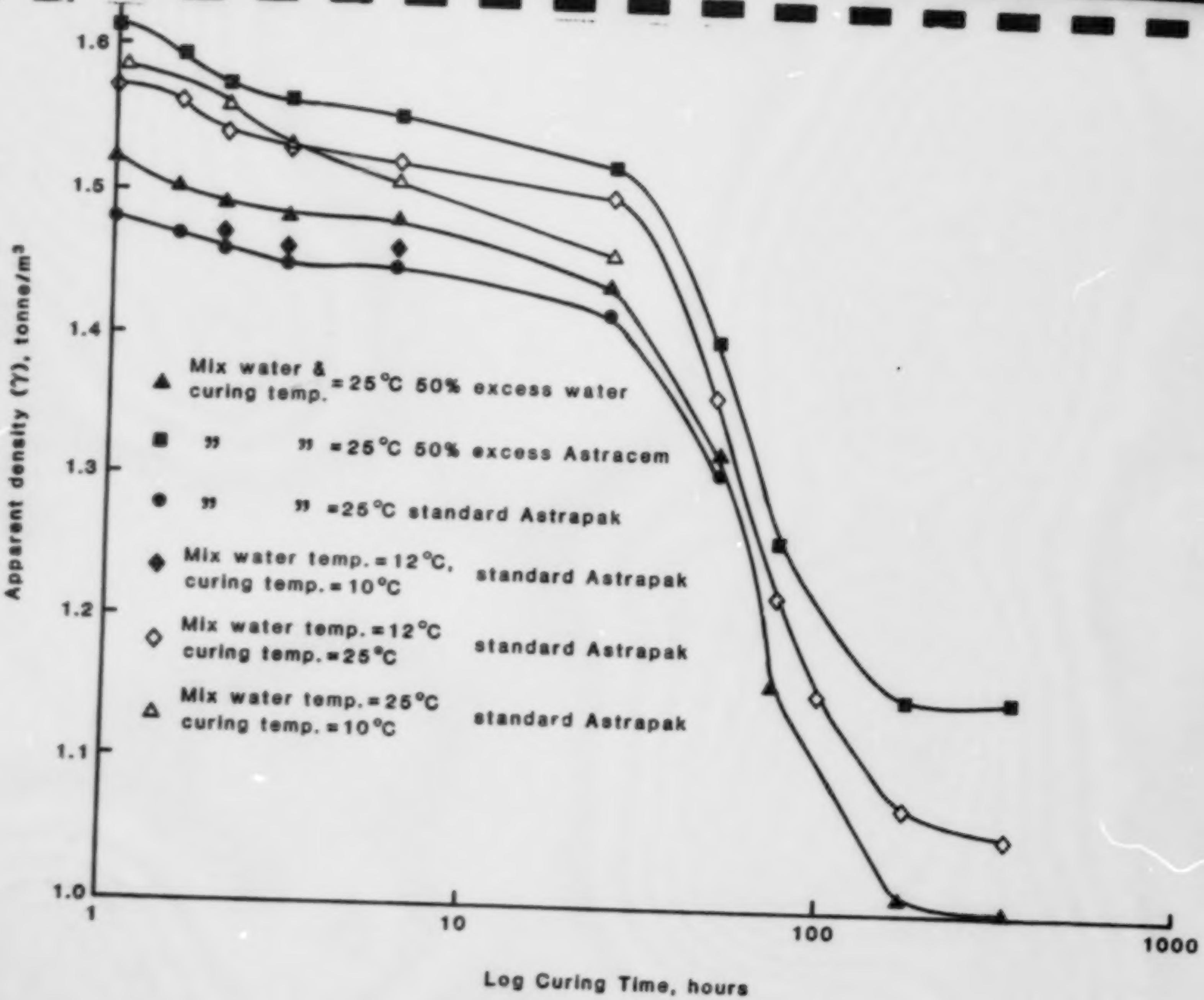
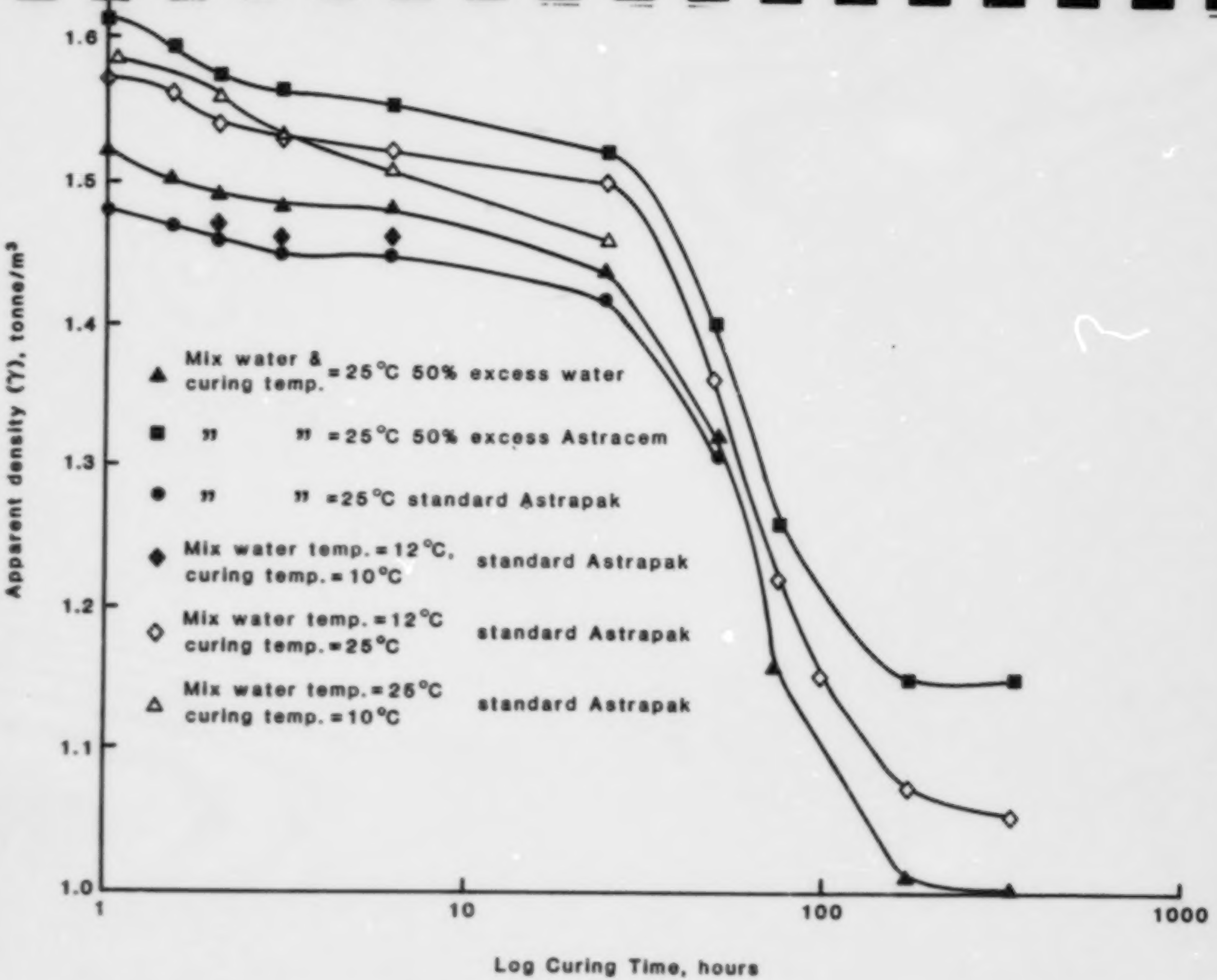

Apparent density (γ), tonne/m³
1.6
1.5
1.4
1.3
1.2
1.1
1.0
1
10
100
1000
Log Curing Time, hours
▲ Mix water & curing temp. = 25°C 50% excess water
■ " " = 25°C 50% excess Astracem
● " " = 25°C standard Astrapak
◆ Mix water temp. = 12°C, curing temp. = 10°C standard Astrapak
◇ Mix water temp. = 12°C curing temp. = 25°C standard Astrapak
△ Mix water temp. = 25°C curing temp. = 10°C standard Astrapak

Table 3 CHANGE IN THE UNCONFINED COMPRESSIVE STRENGTH OF ASTRAPAK DURING CURING TIME (AVERAGE OF 5 TESTS)

Type of			Unconfined Compressive Strength ($_c$), in MPa										
Type of Mixture in sample	Water Temp. (°C)	Curing Temp. (°C)	Curing Time (hours)										
			1	1.5	2	2.5	3	6	24	72	96	168	720
Standard	12	10	0.01	0.20	0.35	0.42	0.52	0.91	1.49	2.04	2.51	3.19	2.73
Standard	12	25	0.23	0.60	0.71	0.75	0.80	1.53	3.03	3.95	4.02	4.11	3.24
Standard	25	10	0.24	0.25	0.26	0.28	0.28	0.30	0.46	0.53	0.71	1.24	1.47
Standard	25	25	0.36	0.57	0.81	0.92	1.08	1.35	2.07	2.69	2.76	2.83	3.40
50% Excess Water	25	25	0.12	0.15	0.16	0.17	0.19	0.28	0.52	0.85	0.73	0.70	0.51
50% Excess Astracem	25	25	0.60	0.80	1.19	1.56	1.57	1.61	2.21	1.65	1.74	2.15	2.46


1.6
1.5
1.4
1.3
1.2
1.1
1.0
Apparent density (γ), tonne/m³
1
10
100
1000
Log Curing Time, hours
▲ Mix water & curing temp. = 25°C 50% excess water
■ " " = 25°C 50% excess Astracem
● " " = 25°C standard Astrapak
◆ Mix water temp. = 12°C, curing temp. = 10°C standard Astrapak
◇ Mix water temp. = 12°C curing temp. = 25°C standard Astrapak
△ Mix water temp. = 25°C curing temp. = 10°C standard Astrapak

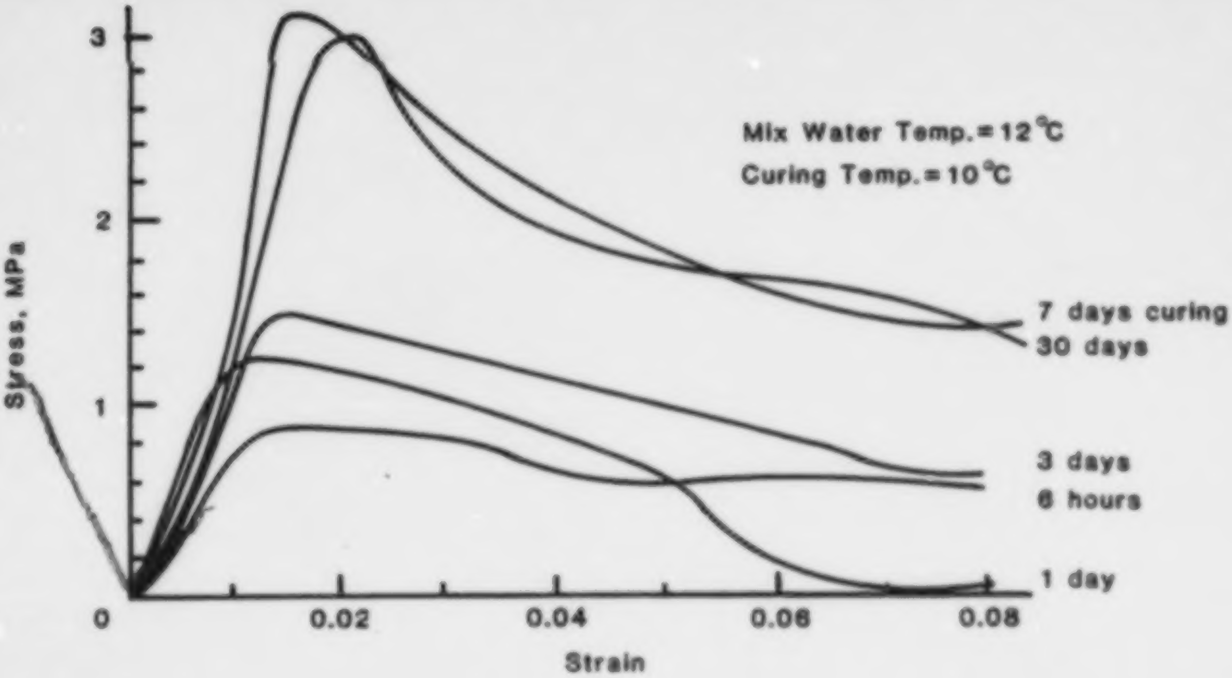


Figure 17 STRESS/STRAIN CURVES FOR THE STANDARD MIX OF ASTRAPAK

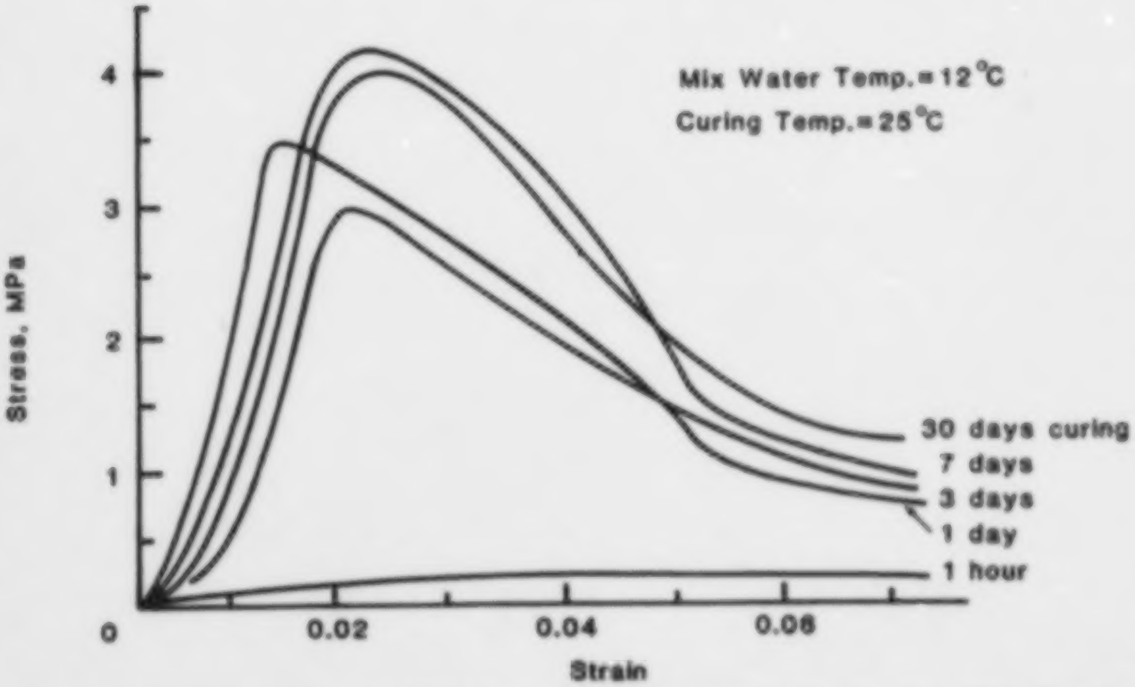


Figure 18 STRESS/STRAIN CURVES FOR THE STANDARD MIX OF ASTRAPAK

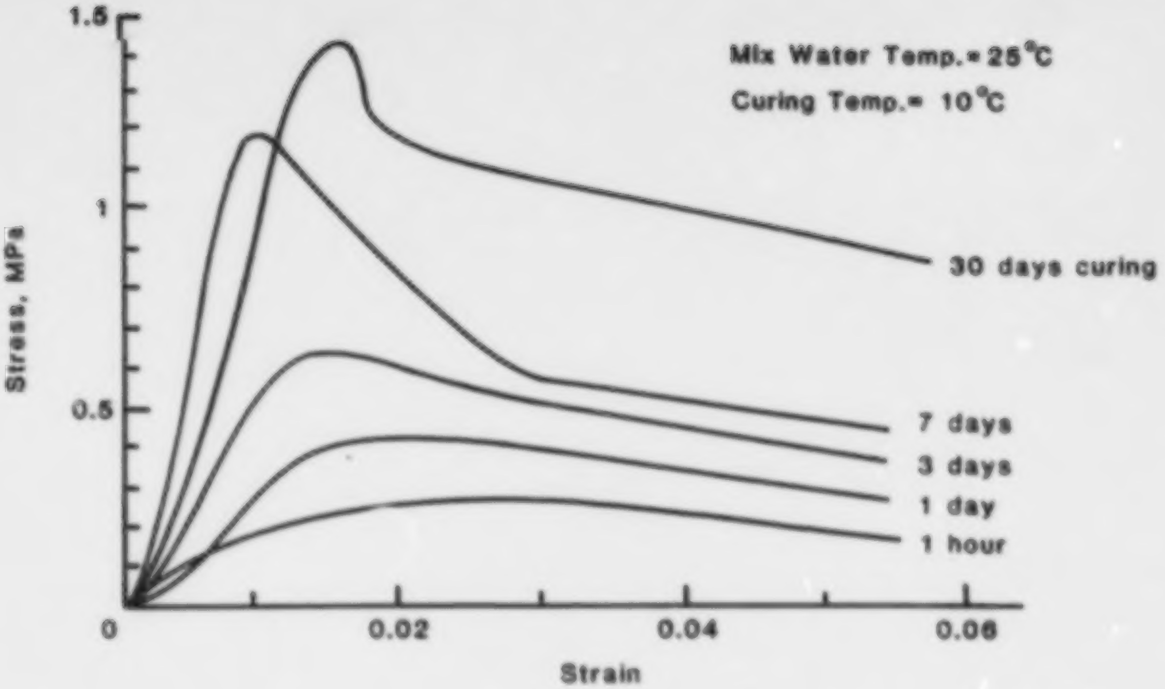


Figure 19 **STRESS/STRAIN CURVES FOR THE STANDARD MIX OF ASTRAPAK**

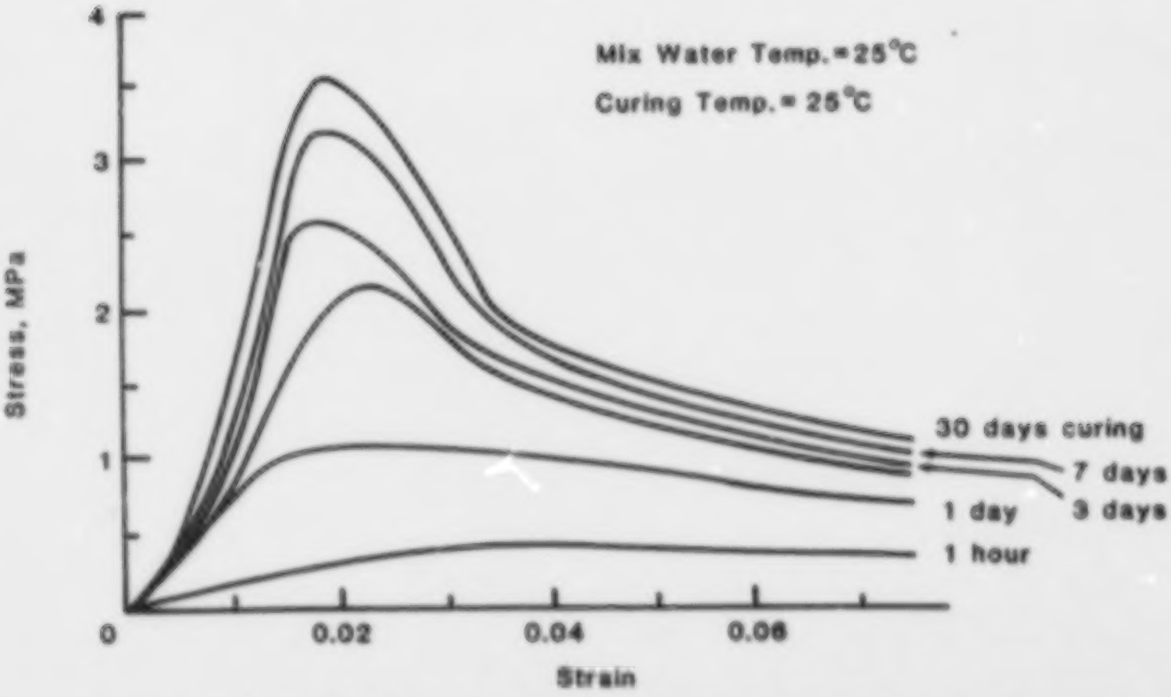


Figure 20 **STRESS/STRAIN CURVES FOR THE STANDARD MIX ASTRAPAK**

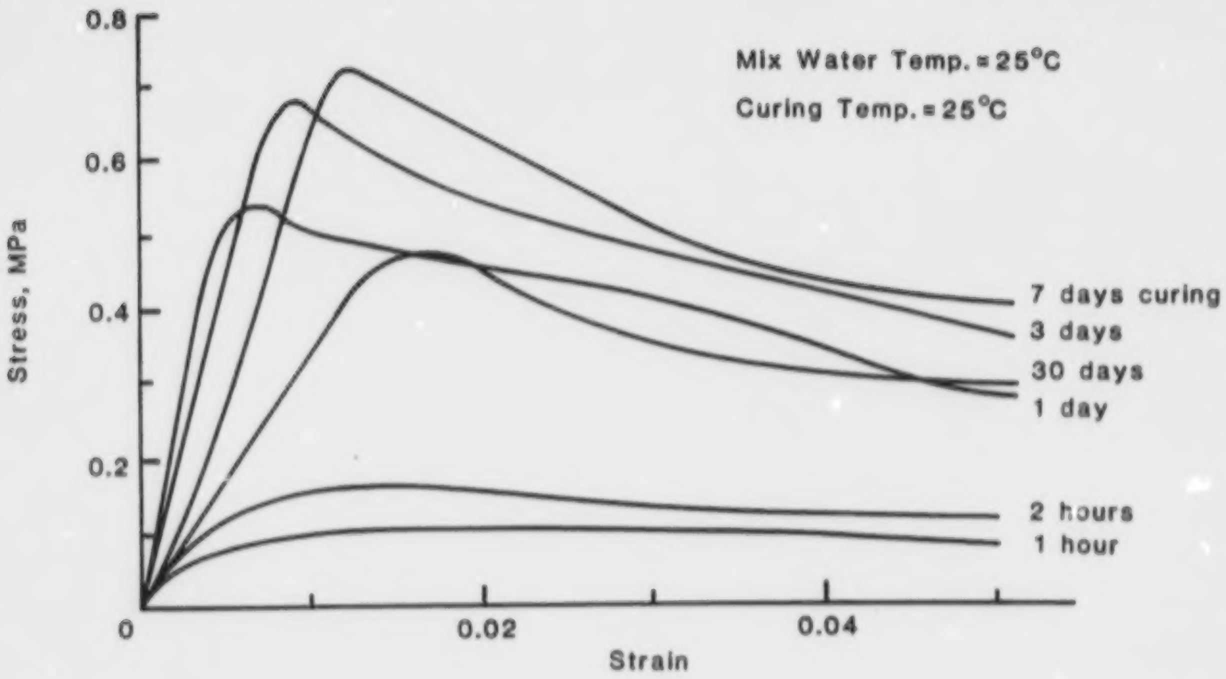


Figure 21 **STRESS/STRAIN CURVES FOR THE ASTRAPAK WITH 50% EXCESS WATER**

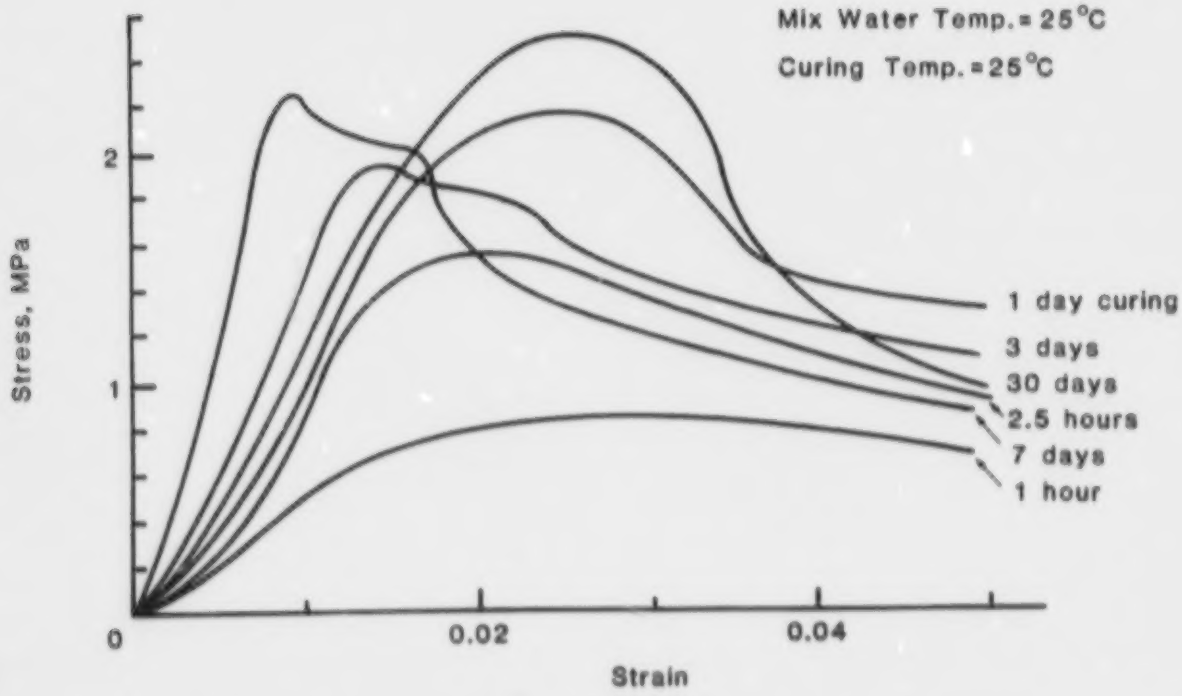


Figure 22 **STRESS/STRAIN CURVES FOR THE ASTRAPAK WITH 50% EXCESS ASTRACEM**

(c) Temperature Measurements of Astrapak Specimens During Curing

The object of this series of tests was to determine the effect of volume or size of samples on the temperature change generated from the chemical reaction within the sample during the curing period of 1 to 168 hours.

The temperature was measured by inserting a pair of prob thermometer in the sample immediately after it is poured in the mold. The prob is fixed in the sample without any movement until it is set hard.

The temperature measurements required for the testing programme were conducted with the aid of National Semiconductor's LM335AH temperature sensors and custom amplifier circuits.

Although the entire circuit was relatively simple in design and application, the resolution of the temperature sensors is greater than 0.1 degrees celcius.

The temperature readings is taken automatically by a transducer temperature measuring device and recorded by data acquisition system. The acquired data was then specified to be processed by the Lotus 1-2-3 for tabulation, (Table 5), mathematical process and graph presentation (Figures 23-29).

(d) Long Term Tests

The samples for this purpose were wrapped in plastic bags and placed the same way as noted in the section 9.2.3. However, the rate of deformation was kept at 0.00001 mm/sec (i.e. 0.00058 mm/min) over 3 days period. The axial and lateral strain devel-

Table 5 **EFFECT OF TYPE AND VOLUME OF ASTRAPAK ON ITS EXOTHERMIC ACTIVITY AND EVALUATION OF THE RELEVANT CONSTANTS**
(average of 5 samples)

Item No.	Pack Material Mixture	Water Temp. (°C)	Curing Temp. (°C)	Pack Volume (v), in cm^3	Curing Time (t). (Hour)	Max. Exotherm (°C)	Total Temperature Change (T) in °C	m	n
				0	0	0	0	23.72	0.06
1	Standard Mix	12	10	125	1.4	6.2	31.2	(Standard error = 3.0%)	
2				1000	1.9	12.4	37.4		
3				3375	2.7	16.3	40.3		
				6,431,250			63.0		
4	Standard Mix	12	25	125	0.2	9.5	34.5	28.00	0.06
5				1000	0.3	17.1	42.1	(Standard error = 3.0%)	
6				3375	0.4	21.0	46.0		
				6,431,250					
7	Standard Mix	25	10	125	0.8	4.5	24.5	17.25	2.4%
8				1000	1.1	9.8	29.8	(Standard error = 4.2%)	
9				3375	1.7	14.5	34.5		
				6,431,250					
10	Standard Mix	25	25	125	0.9	10.0	35.0	29.48	0.06
11				1000	2.4	21.6	46.6	(Standard error = 9.2%)	
12				3375	2.8	24.5	49.5		
13				15625	4.4	30.8	55.8		
				6,431,250					
14	Astrapak + 50% Excess Water	25	25	125	0.4	3.7	28.7	20.86	0.07
15				1000	1.8	9.5	34.5	(Standard error = 2.7%)	
16				3375	2.6	15.0	35.7		
				6,431,250			61.0		
17	Astrapak + 50% Excess Astracen	25	25	125	1.3	9.2	44.2	38.98	0.04
18				1000	1.6	25.8	50.8	(Standard error = 4.9%)	
19				3375	2.1	30.5	54.5		
				6,431,250			71.0		

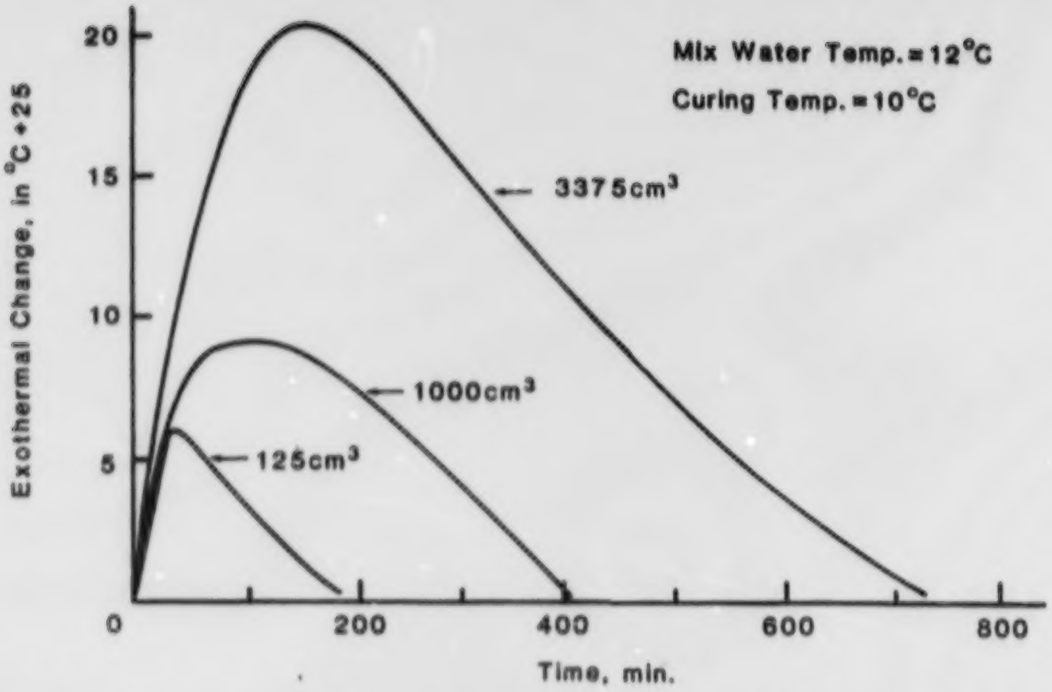


Figure 23 **EFFECT OF PACK SIZE ON THE EXOTHERMIC ACTIVITY OF THE STANDARD ASTRAPAK**

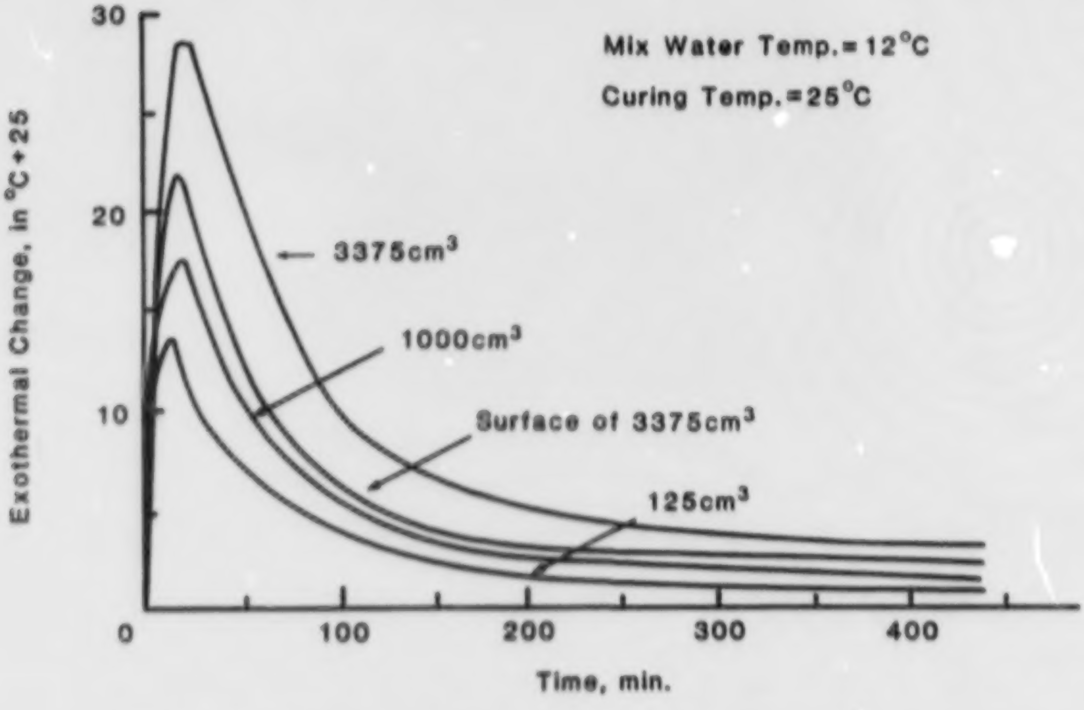


Figure 24 **EFFECT OF PACK SIZE ON THE EXOTHERMIC ACTIVITY OF STANDARD ASTRAPAK**

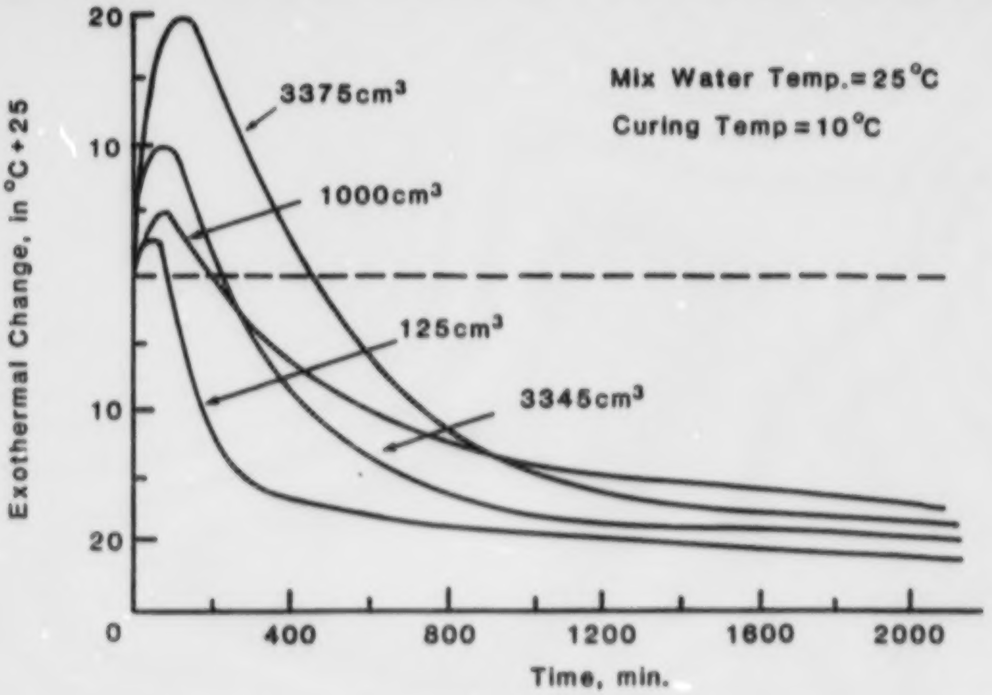


Figure 25 **EFFECT OF PACK SIZE ON THE EXOTHERMIC ACTIVITY OF THE STANDARD ASTRAPAK**

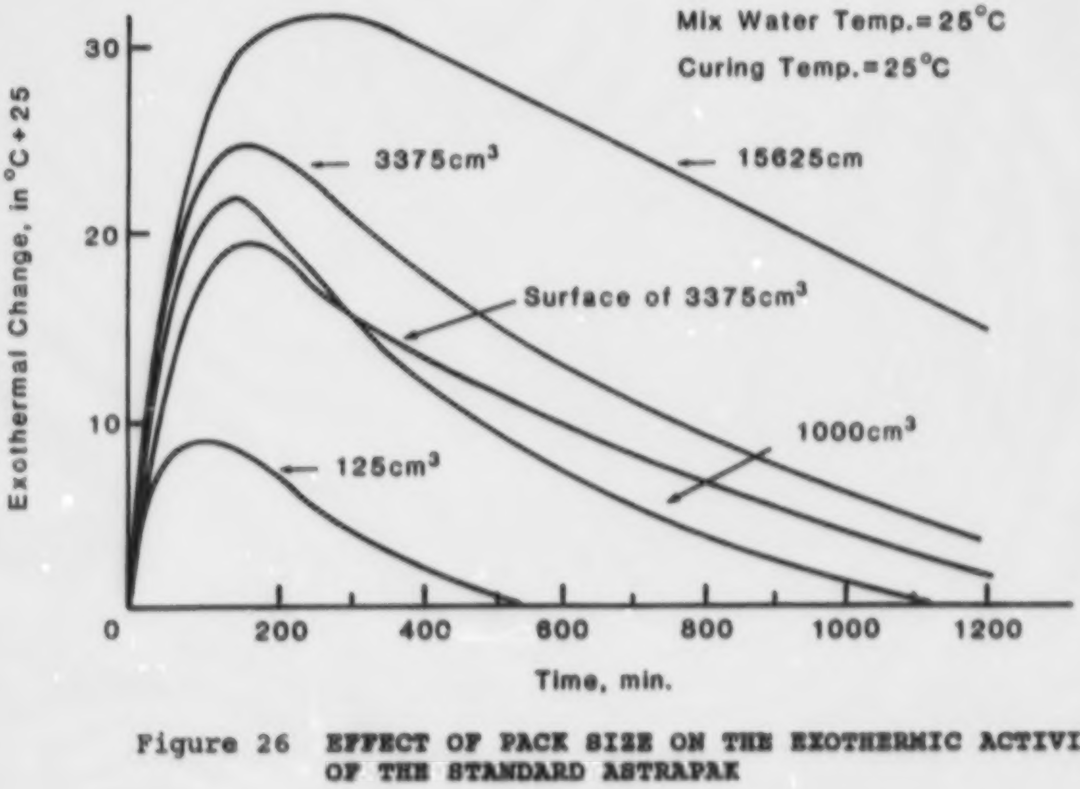


Figure 26 **EFFECT OF PACK SIZE ON THE EXOTHERMIC ACTIVITY OF THE STANDARD ASTRAPAK**

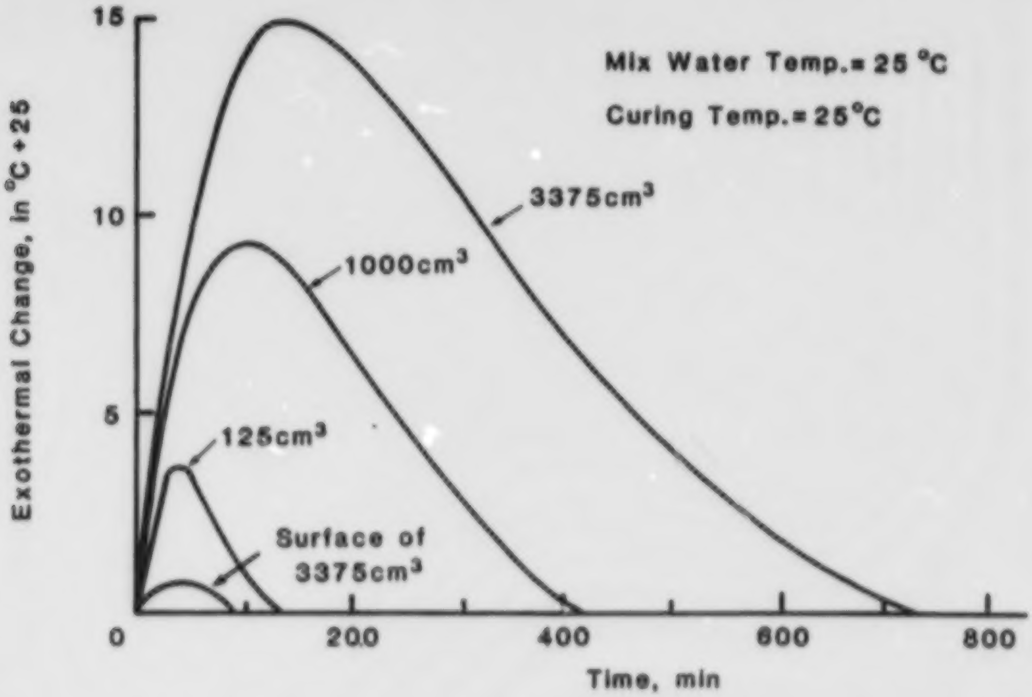


Figure 27 **EFFECT OF PACK SIZE ON THE EXOTHERMIC ACTIVITY OF THE ASTRAPAK WITH 50% EXCESS WATER**

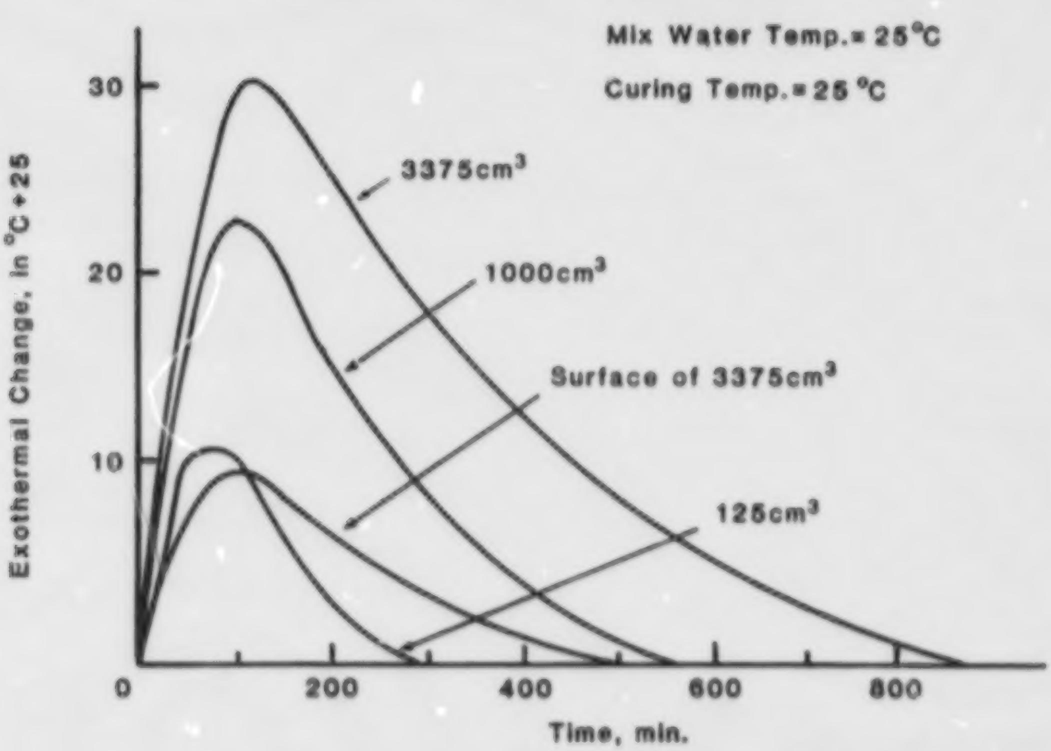
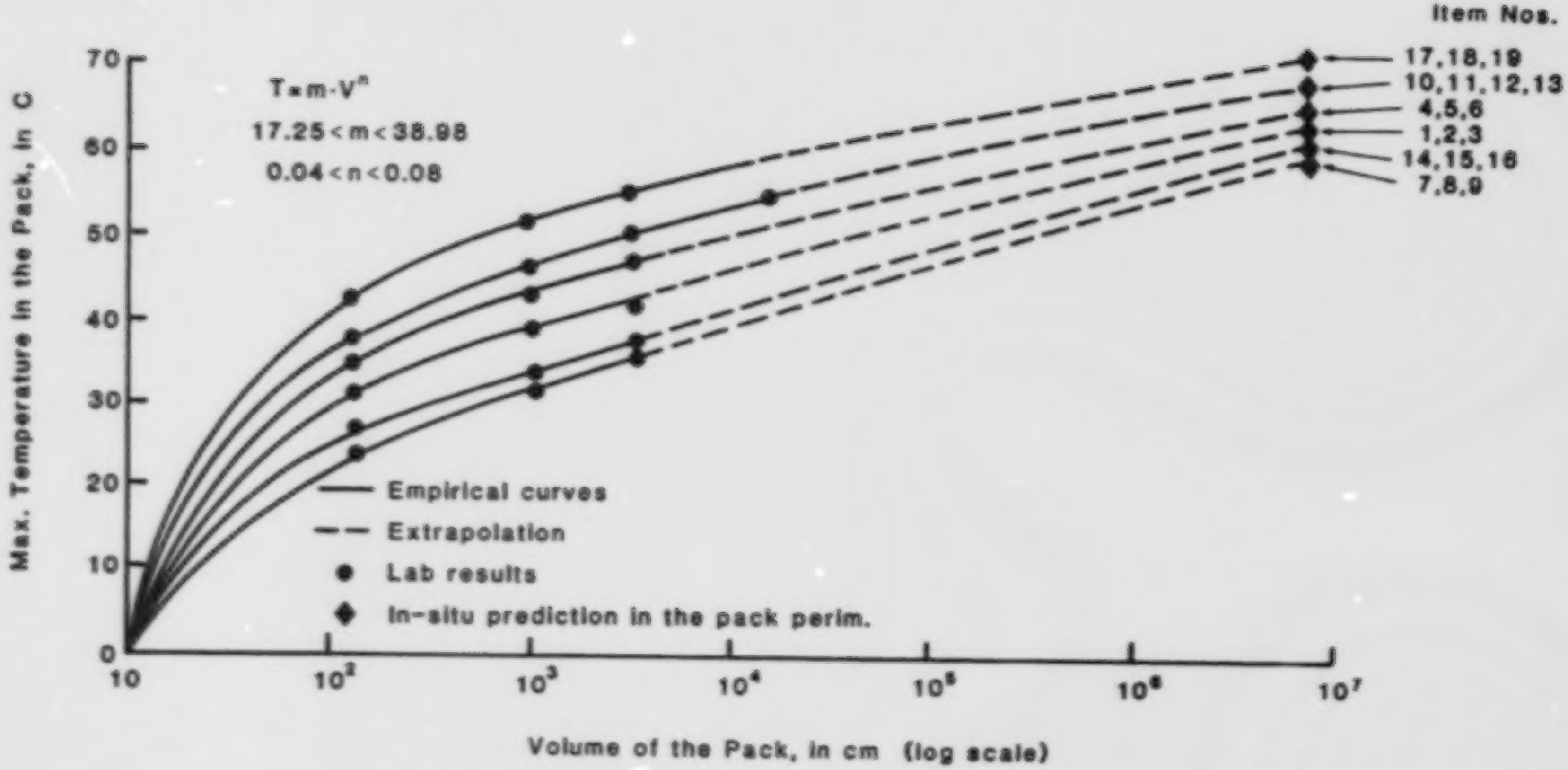


Figure 28 **EFFECT OF SIZE ON THE EXOTHERMIC ACTIVITY OF THE ASTRAPAK WITH 50% EXCESS ASTRACEM**


Max. Temperature in the Pack, in C
70
60
50
40
30
20
10
0
10
10^2
10^3
10^4
10^5
10^6
10^7
Volume of the Pack, in cm (log scale)
$T = m \cdot V^n$
$17.25 < m < 38.98$
$0.04 < n < 0.08$
Empirical curves
Extrapolation
Lab results
In-situ prediction in the pack perim.
Item Nos.
17,18,19
10,11,12,13
4,5,6
1,2,3
14,15,16
7,8,9

opment in the sample and the axial stress was noted. Average curve for the results is shown in Figure 30.

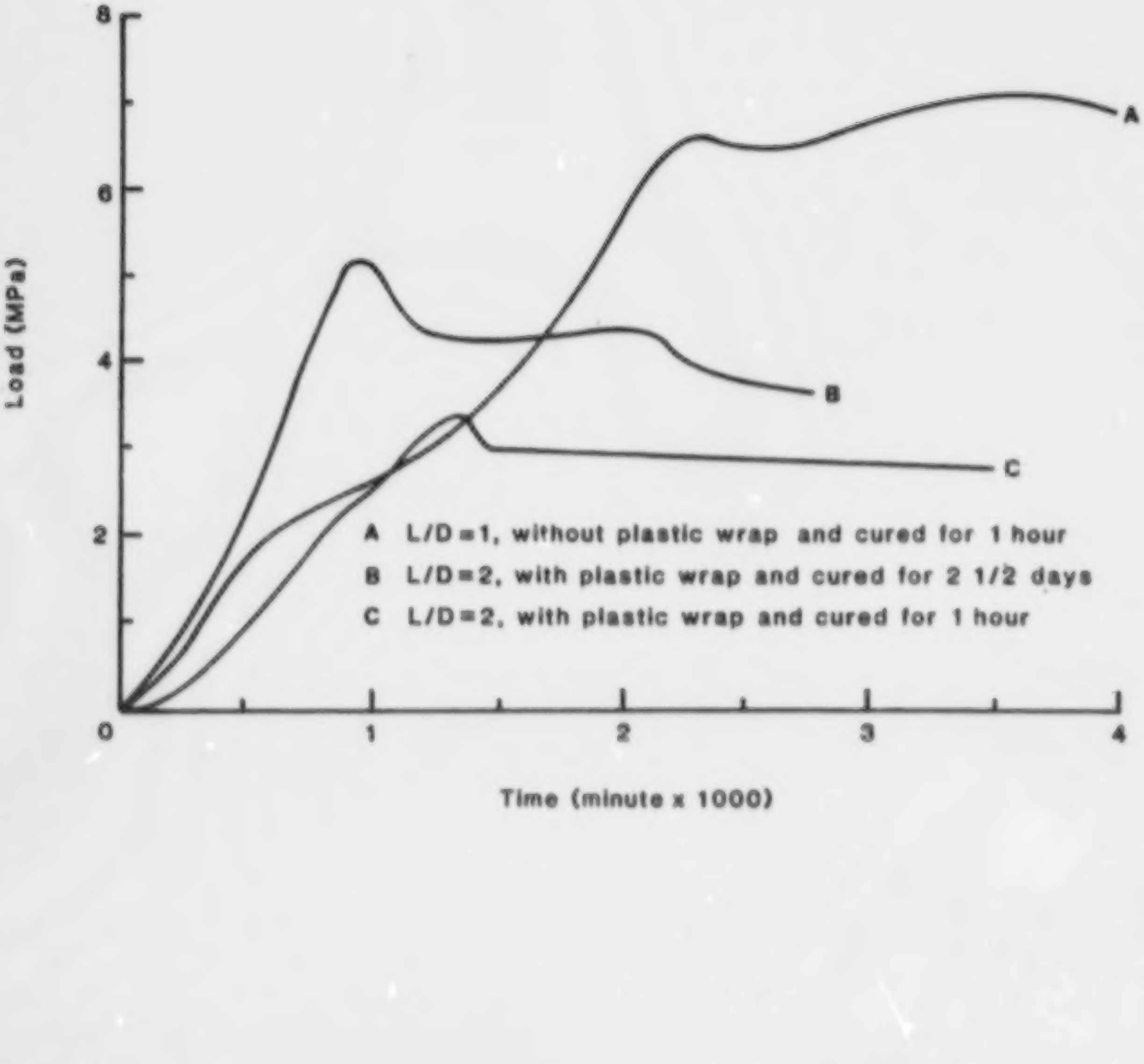


Figure 30 LONG TERM BEHAVIOUR OF VARIOUS ASTRAPAK SAMPLES OF CONSTANT RATE OF DISPLACEMENT (0.0006 mm 1 min.)

6 - ANALYSIS OF THE ASTRAPAK TEST RESULTS

6.1 - CHANGE OF DENSITY (W) WITH CURING TIME (t)

In this test change of density in the Astropack samples was considered to be due to reduction of water in the sample. This was due to 2 phenomena:

(a) Creation of heat due to chemical action in the mixture during curing which tend to vaperize the water and moisture in the sample. The vapor is then directed towards the surface of the sample.

(b) Exchange of water and vapour between the sample and its surroundings. In this context, if the environment surrounding the sample is saturated with moisture and/or the percent moisture of the sample is less than that of the surrounding, then the rate of transfer of moisture from sample to the surrounding environment is reduced proportionally. This general trend was seen in various types of Astrapack cured in the room temperature of 25°C (Figure 15). However, in the 10°C temperature environment, the samples have shown constant density after 24 hours of curing time, which was due to the part (b) described above. As can be seen from Figure 15, the rate of density change was markedly different after 24 hours of curing time for all the sample types. Hence the first 24 hours is a crutial curing time to make a good Astrapak with least flows . Lower water temperature in the mix and/or excess water had an adverse effect on the density and rate of curing. In general, the mix with 50% extra Astracem had the highest density and the standard mix cured at 25°C had the lowest density throughout the test. The rate of density change was gradually declined with increase in the curing time.

6.2 - FACTORS INFLUENCING THE STATIC MODULUS OF ELASTICITY (E) OF THE ASTRAPACK

A general increase in the E-value of all the sample types under stress was noted with increase in the curing time (Figures 17-22). However, the rate was decreased by 60% on average when 50% excess water was introduced to the mixture, and there was a 70% decrease in the E-development rate when the curing temperature was decreased by 17%. Typical curves of the modulus of elasticity vs. compressive strength of astrapak having different mixture, water temperature and curing temperature is shown in Figures 17-22. It can be seen that the pack exhibit elasto-plastic behaviour, except when 50% excess Astracem is added to the standard mixture at water and curing temperature of 25°C, when the behaviour tends towards elastic, in short duration test.

6.3 - CHANGE IN UNCONFINED COMPRESSIVE STRENGTH OF THE ASTRAPAK (σ_c) WITH CURING TIME

The strength in all types of samples tested increased relatively rapid by the curing time during the first day. After this period the rate of strengthening was in decreasing order (Figure 17). In two cases where 50% excess water and also where water at 50% lower temperature was used, the strength tended to decreased after 4 days by reaching a peak of 0.7 and 4 MPa, respectively.

In all cases the strength development vs. curing time for the Astrapak was below the manufacturer's suggestion.

The samples with 50% excess Astracem reached σ_C = 0.6 MPa within an hour. Whereas the samples with 50% excess water content reached σ_C = 0.12 MPa in one hour of curing time. Lower water and curing temperatures had an adverse effect on the σ_C of the Astrapak, as is shown in Figures 18-22.

6.4 - EFFECT OF THE SAMPLE SIZE ON THE TEMPERATURE CHANGES DURING CURING

For all the samples, the larger the dimensions of the pack material, the higher was the temperature evolved during their curing. This effect was observed underground to such extent that it would interfere with the ventilation and working environment of the section where the pack was employed. The extent of the effect was thought to accelerate the spontaneous combustion of the coal in the vicinity. Figures 23 to 28 show the trend of exothermic activity of Astrapk samples and its relation with sample size. The maximum temperature in the molded samples are reached within one to three hours of their placement. After which the total temperature gradually drops towards the ambient. The temperature measurements in the above noted graphs was measured from the ambient value of 25°C upwards. Therefore, all the values on the curves should be added to 25°C for total temperature evaluation. The general rate of max. temperature increase with size is noted in Table 5 and shown in Figure 29 and expressed mathematically as follows:

$$T = m.V^n$$

where:

T = Max. internal temperature of curing Astrapak, in ^oC.

m = Constant related to the mix. water and curing, or ambient, temperatures

= Between 12.4 to 30.0. For the standard Astrapak at low temperatures it averages to about 12.4 to 20.9.

n = Constant related to the free sides of the pack for heat transfer between the pack and the mine air and the pack dimensions.

= Between 0.08 to 0.12. For the standard Astrapak at low temepratures it averages to about 0.10 to 0.13.

Introduction of 50% excess water and 50% excess Astracem caused lowest and highest temperature at any incremental curing time, respectively. This behaviour is evident from Figures 23 to 29, at various increments of time.

Low water temperature and curing temperature also caused lower rate of temperature increase with size which in turn increased the curing time, Figures 25 to 29. However, at 11 of the above noted conditions and mixture preparations the size effect was directly but non-linearly proportional to the max. temperature creation during the curing.

6.5 - LONG TERM EFFECT OF THE ASTRAPAK ON ITS UNCONFINED COMPRESSIVE STRENGTH (σ_c)

The aim was to improve understanding of the Astrapak behaviour under constant rate of displacement, underground. For this reason a displacement rate of 0.0006 mm/min was chosen. This provided an average rate of stress equivalent to 0.0012 MPa/min upto failure under unconfined condition.

Three different categories of standard mix made at water and curing temperatures of 25°C was tested. The test results are shown in Figure 30. It appears that packs with lower L/D ratio exhibit higher long term σ_c. Also longer initial curing before application of the load on the pack will result to a higher failure strength. This has a practical application for in situ mixing, pouring and setting time before the strata load is acted on the packs.

It should be noted that after the failure, the pack still does have load resistance to collapse much the same way as a pillar would have underground. This mechanism can be clearly seen from Figure 30.

6.6 - EFFECT OF WATER ON THE COMPRESSIVE STRENGTH (U.C.S.) OF ASTRAPAK

As noted previously, ratio of the Astrapak constituents for the standard mix was:

Astrabent : Astracem: Water = 1 : 1 : 5.01

Increase in the water content causes decrease in the U.C.S. of the pack, proportionately. Therefore, a decrease in the water content would increase the U.C.S. of the pack.

It was found that the following ratio to provide the optimum compressive strength of the order of 2 to 4 times higher than the standard mix, depending on the curing time. These strengths are much higher than needed in the pack for coal measures.

Astrabent: Astracem: Water = 1 : 1 : 2.9

Test results for the latter mix is noted in Table 6 and shown in Figures 31 to 33.

Advantages of the later mix ratio are as follows:

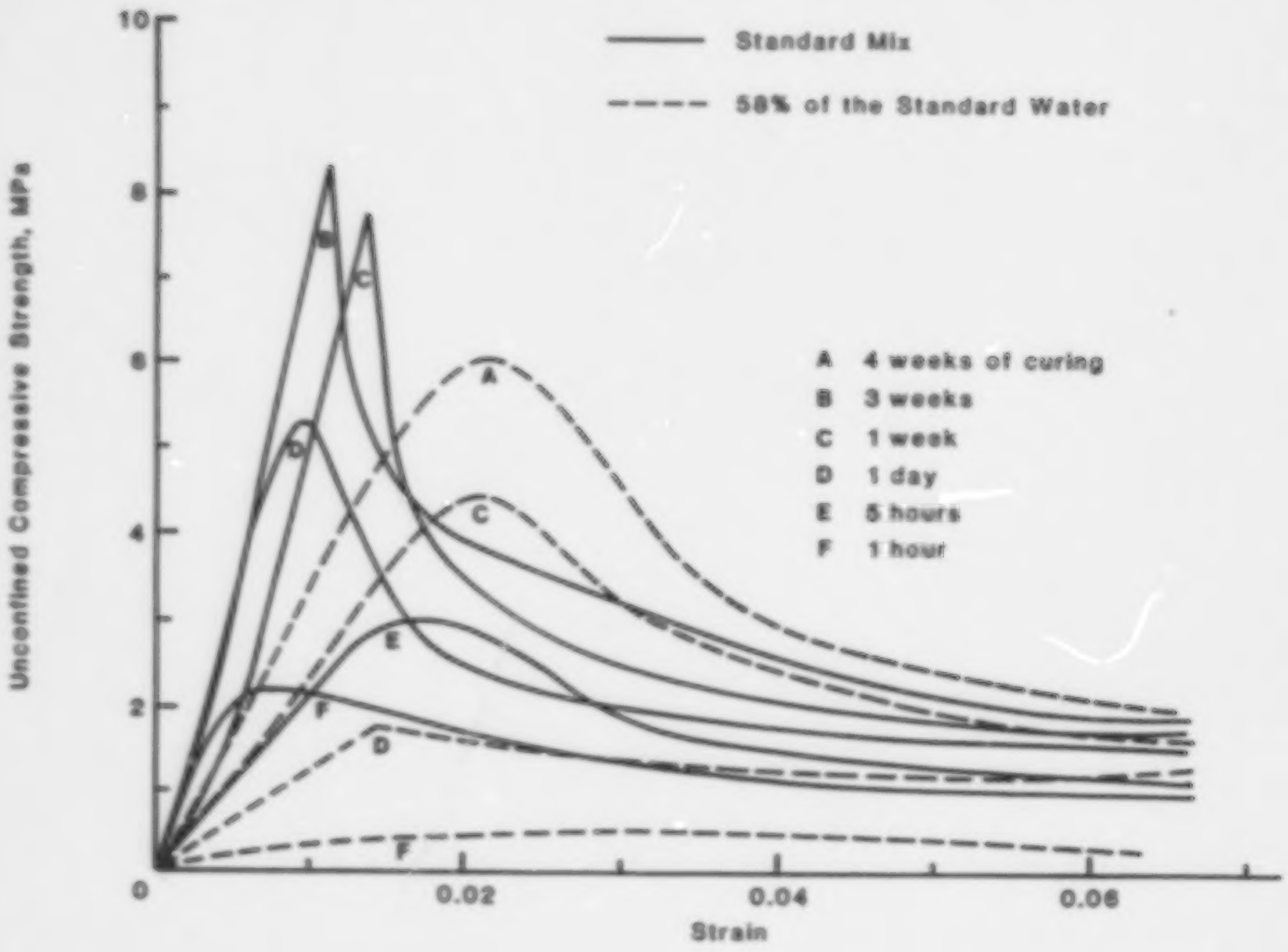
(a) Various mixes are pumpable by long piping to the pack site as the standard mix is.

(b) The mix does not gel so quickly. Therefore placing of the final mix is practical.

(C) The reduction in volume of the Astrapak between the standard mix and the latter mix can be overcome by addition of mine tailing and/or waste rocks of -6 mm in size in the form of slurry to the Astrabent mix. This will give extra volume to the pack while decrease the U.C.S. of the Astrapak to a lower pre-selected value by proportioning the ratio of the mixes. Work is on hand to formulate appropriate and economically feasible ratios for the latter mix. In this way some of the waste rocks can be utilized and the transportation cost to move them out of the mine to the waste dump reduced.

Table 6 **PHYSICAL AND MECHEMICAL PROPERTIES FOR THE ASTRAPAK WITH 50% OF THE STANDARD WATER**

Properties	Temperature (^0C)	Curing Time before Test				
		1 Hr.	5 Hrs.	1 Day	1 Week	3 Weeks
Density (gr/cm^3)	WT = 24 CT = 24	1.40	1.42	1.39	1.40	1.44
	WT = 12 CT = 24	1.41	1.41	--	1.41	--
	WT = 12 CT = 12	1.41	1.40	1.40	1.42	1.41
Ultrasonic Velocity (mm/ /- sec)	WT = 24 CT = 24	1.73	1.71	1.78	1.90	1.91
	WT = 12 CT = 24	1.64	1.80	--	1.99	--
	WT = 12 CT = 12	1.67	1.77	1.80	1.90	2.02
Unconfined Compressive Strength (MPa)	WT = 24 CT = 24	2.23	3.06	5.19	7.77	8.35
	WT = 12 CT = 24	1.21	3.33	--	9.75	--
	CT = 12	WT = 12	1.29	3.58	3.84	6.79
Modulus of Elasticity (MPa)	WT = 24 CT = 24	426.9	199.1	370.7	431.7	597.7
	Wt = 12 CT = 24	232.7	320.2	--	870.5	--
	WT = 12 CT = 12	215.0	596.7	640.0	772.3	902.7


Unconfined Compressive Strength, MPa
10
8
6
4
2
0
0.02
0.04
0.06
Strain
Standard Mix
58% of the Standard Water
A 4 weeks of curing
B 3 weeks
C 1 week
D 1 day
E 5 hours
F 1 hour
A
B
C
C
D
D
E
F
F

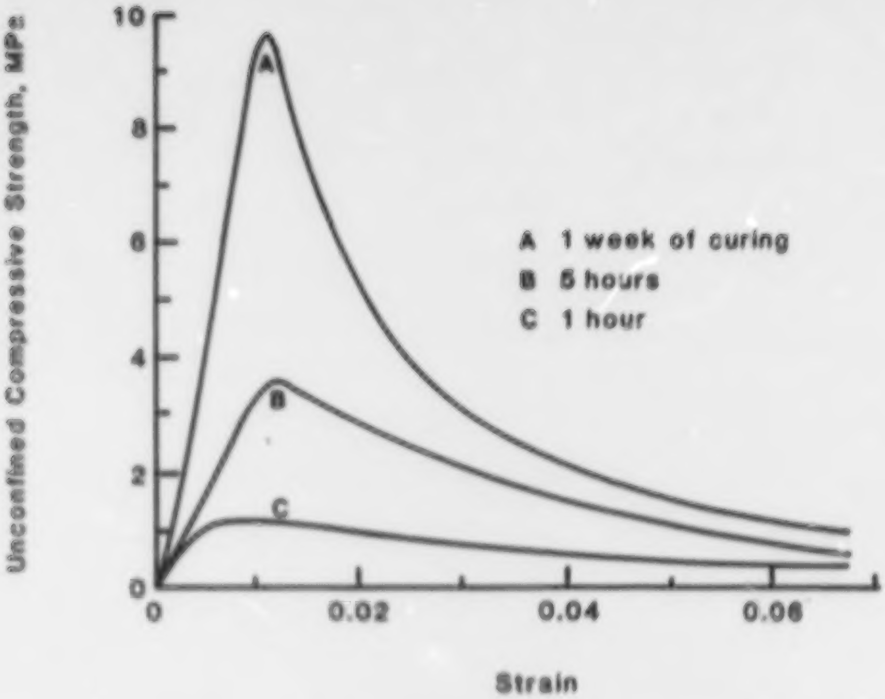


Figure 32 COMPARISON OF STRESS/STRAIN CURVES FOR THE ASTRAPAKS WITH 58% OF THE STANDARD WATER MADE AT W.T. = 12°C AND C.T. = 24°C

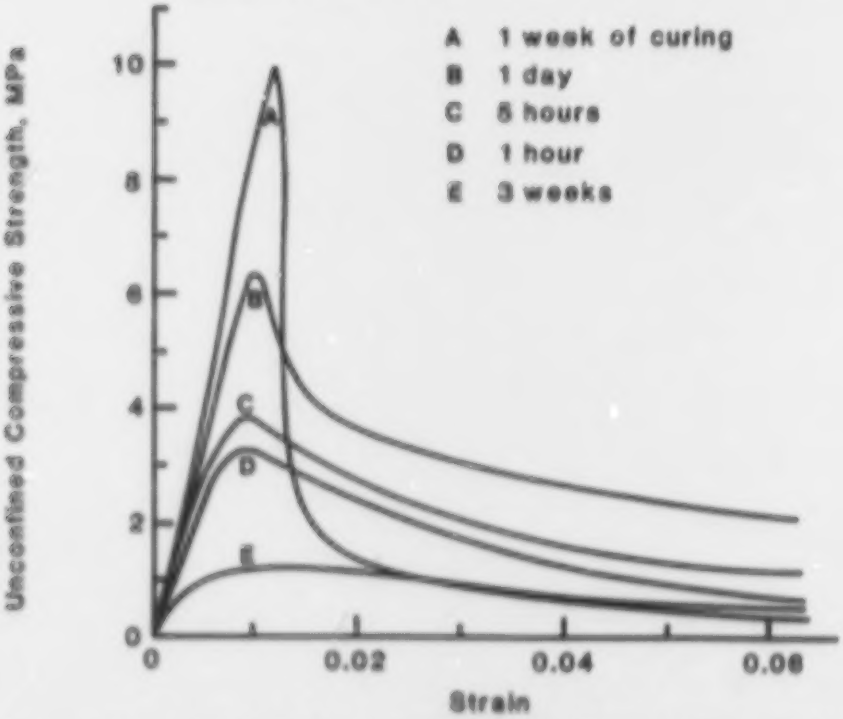


Figure 33 COMPARISON OF STRESS/STRAIN CURVES FOR THE ASTRAPAKS WITH 58% OF THE STANDARD WATER MADE AT W.T. = C.T. = 12°C

The disadvantage of utilizing the waste rock and or the mine tailings as the pack filling material is that an additional cost may be incurred for a small cycle of crusher classifier - 6 mm, larger mixer and pump for the waste rock plus Astrabent slurry as shown schematically in Figure 34.

6.7 - COMPARISONS BETWEEN AQUAPAK, TEKPAK AND ASTRAPAK

(a) The material cost of Tekpak and Astrapak is 17% less than that of the Aquapak.

(b) Tekpak and Astrapak need higher % of water than the Aquapak.

(c) Aquapak performs better in cool environment, whereas Tekpak and Astrapak require warmth for correct curing.

(d) Aquapak has a more yielding ability under load than the other two pack types. However, its strength development per curing time is relatively lower than the Tekpak and Astrapak (Figure 35).

(e) In long term, as the pack's strength rather than yielding properties become more important Aquapak has the greater strength. Once again, underground experience leads to the conclusion that the Aquapak strength is adequate for the task. It is likely that the strength displayed by Tekpak at 7 days, given sufficient pack dimensions, would also prove adequate, Whittaker et al [4]. Further deterioration in the strength of the material might cause failure of the pack. At this stage of the pack's life restitution of its strength by further curing is an extremely remote possibility.

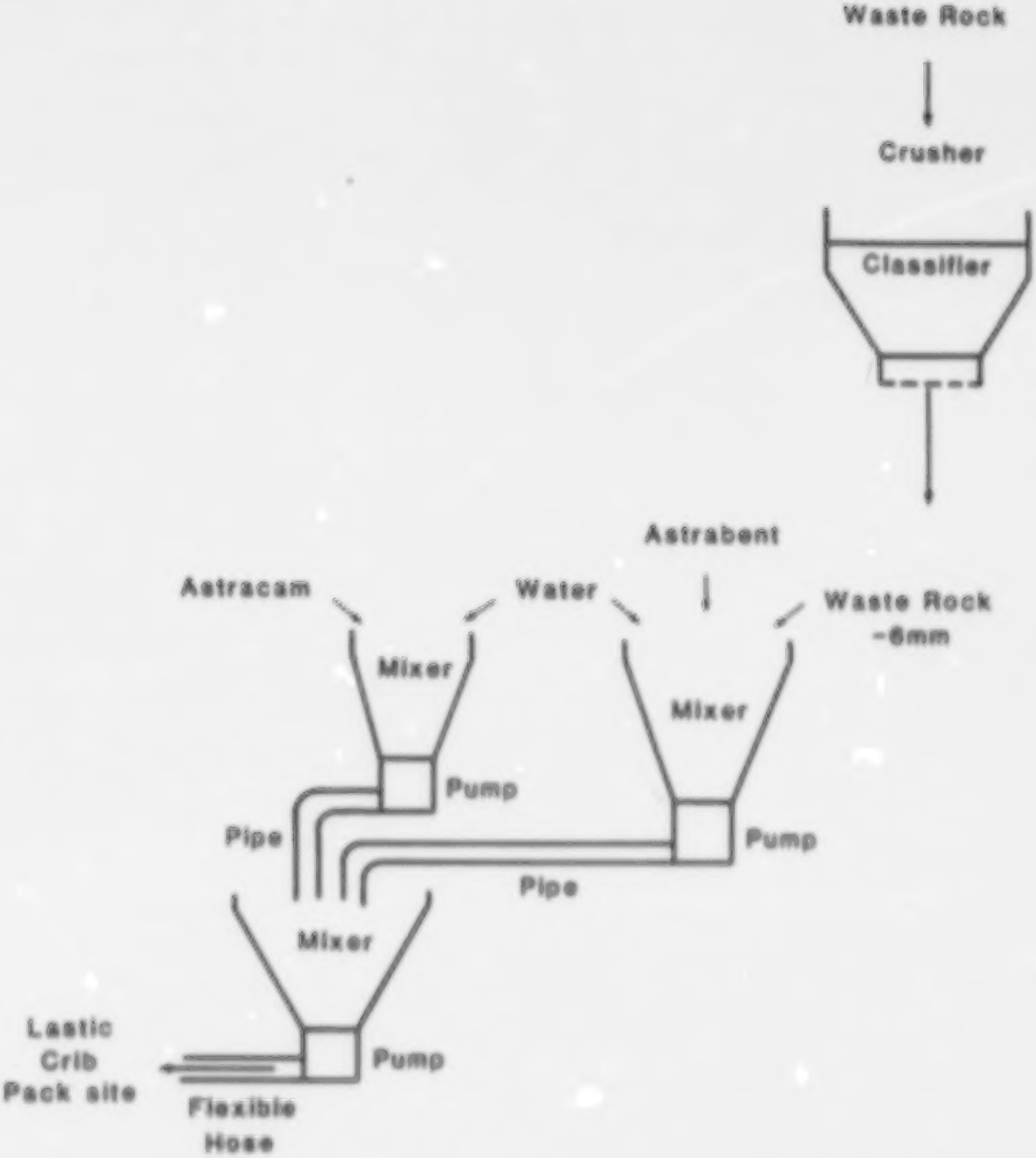

Waste Rock
Crusher
Classifier
Astrabent
Astracam
Water
Waste Rock
-6mm
Mixer
Mixer
Pump
Pump
Pipe
Pipe
Mixer
Lastic
Crib
Pack site
Pump
Flexible
Hose

Figure 34 SCHEMATIC OF THE ASTRAPAC/WASTE ROCK ARRANGEMENT

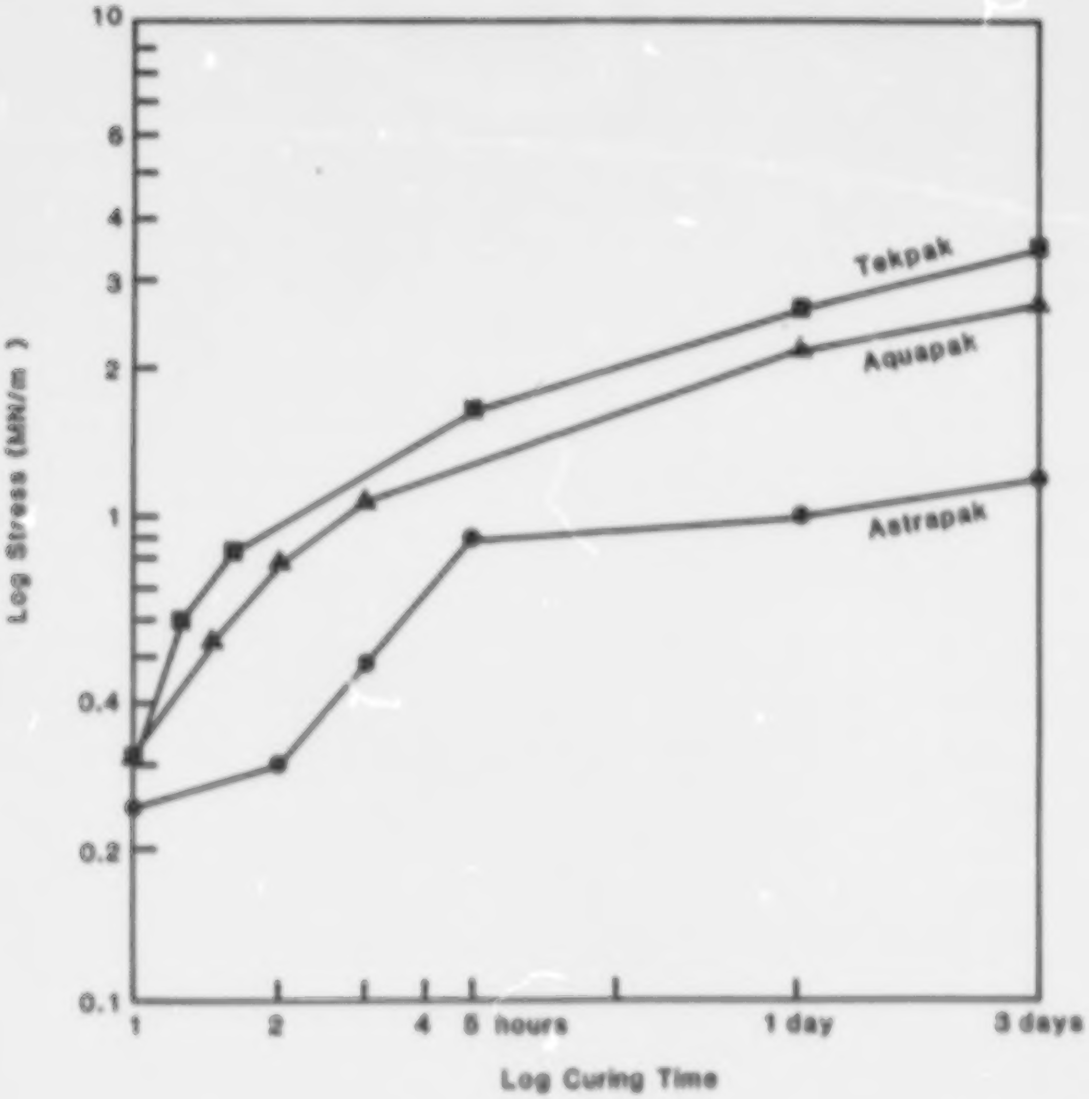


Figure 35- Comparison of the strength development for the standard mixes of Aquapak, Tekpak, and Astrapak.

The form of the long term graphs for Aquapak is more ideal than that for the Tekpak, the rate and amount of stress change being lower. Further tests along these lines are necessary to clarify the situation.

(f) All three materials displayed the ability to develop full strength despite a degree of failure during their early life. This is a very valuable property.

Tekpak and Astrapak are very similar in their mechanical behaviour.

Tekpak improves its strength properties with a 15% increase in Tekbent. 30% and 50% increases of Tekbent dramatically improve the plasticity of the material but markedly reduce its strength. Further investigations may reveal that Tekpak behaviour can be improved upon by varying the proportion of the constituents.

Astrapak would have a higher strength when 50% Astracem is added to its standard mix. However, its cost would increase proportionally.

(g) Cost breakdown and unconfined compressive strength comparison for various monolithic pack materials is noted in Table 7.

In comparison between the three pack materials, followings are definite conclusions :

(i) Incorrect mixture proportions, and excess water content will slow the pack curing time, slowing the pack cycle and therefore cause insufficient long term pack strength development. Excess solids (in particular cement) will substantially increase the

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS

(AS OF 1985 PRICES)

Pack Type	Material Costs (CAN$/m³ of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Aquapak	Aquabent	50	12.50		0.52	1.30	4.00	5.00
	Aquacem	450	95.86	108.36				
Thyssen	Coal	500		0.65	0.40	0.95	1.34	
	Flowmat	16	2.16					
	Packbind	290	64.44	100.17				
Warbret	Coal	500	33.57		0.07	2.00	3.10	4.90
	Flowmat	16	2.16					
	O.P.C. (Bulk)	290	23.53					
	TEA 51	30	24.03	83.29				
Tekpak	Tekbent	102	40.85		0.87	2.50	4.0	4.2
	Tekcem	102	57.95	98.80				
Astrapak	Astrabent	103	37.75		0.5	1.4	4.4	4.5
	Astracem	103	52.96	90.71				

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/m³ of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Flashpak	Fly Ash	921	27.63		0.02	0.9	4.1	9.3
	O.P.C. (Bulk)	255	41.31					
	$FeSO_4$	100	22.50					
	Polbent	18	5.94	97.30				
	Fly Ash	930	30.63		0.40	1.6	4.4	8.2
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	65	26.62					
	Polbent	95	31.67	128.23				
	Fly Ash	930	30.63					
	O.P.C. (Bulk)	255	41.31					
	$Al_2(SO_4)_3$	63	16.29	88.23				
Packblend	Limestone*	2000	73.59		0.78	1.53	6.80	9.80
	Rapid Setting Cement (Bulk)			73.59				

Table 7 COST BREAKDOWN AND U.C.S. COMPARISON FOR VARIOUS MONOLITHIC PACK MATERIALS (Cont'd)

Pack Type	Material Costs (CAN$/m^3 of pack)				Uniaxial Compressive Strength (MPa)			
	Materials		Costs excluding transport and crib bag					
	Name	Weight (kg)	Individual	Total	2 Hours	1 Day	7 Days	28 Days
Anpak	*Synthetic Anhydrite (Bulk)*	2000	39.79		0.26	6.1	21.0	28.0
	Accelerator	20	8.65	48.44				
	Natural Anhydrite (Bulk)	2000	19.90		0.05	0.8	14.5	20.3
	Accelerator	56	24.22	44.12				

density and initial setting and packing costs. The rate of hardening per curing time was thereafter slower, comparatively.

(ii) Density change in the pack materials can be expected to be rapid during the first 2 to 3 hours of curing time. After which the rate of change would gradually decrease with time until relative stability is reached in about 7 days.

(iii) Packs prepared water of 25°C and cured at ambient temperature of 25°C exhibit rapid to instantaneous gelling properties. Similar pack materials prepared at lower water temperatures of 10 to 12°C exhibit no gelling properties and no strength development after approximately one hour.

(iv) Water and curing temperatures are more influential factors than the excess cement for the strength of a pack, over long periods. However excess cement cause more rapid initial hardening within the first 2-hours of curing time.

(v) The rate of increase in modulus of elasticity of Astrapak is adversely affected by 70%, when a decrease of 17% occurred in the curing temperature and/or 50% excess water was introduced to the mixture.

(vi) The lower the L/D ratio or the ratio of pack height to the pack base dimensions, the higher would be its unconfined compressive strength.

(vii) Longer initial curing before application of the strata load on the pack would result to a higher long term pack strength.

7 - ANHYDRITE PACKS (ANPAK)

Anhydrite packs was developed for use in German coal mines in 1964 with the first major trial taking place at the Holland Colliery. Anhydrite was introduced as a substitute material for pack formation and has since grown in popularity, especially in German longwall mining operations. Current Ruhr coalfield anhydrite consumption exceeds 300,000 tons. Anhydrite packing systems were developed to :

(a) Minimize the quantities of expensive wooden chocks required for packing.

(b) Reduce costly pack material transportation and installation time and manpower.

(c) Overcome the face supply problems associated with wood chock packing.

(d) Improve face-end ventilation efficiency by reducing air leakage through the waste area.

(e) Reduce the risk of spontaneous heating in the goaf.

The anhydrite packing system relies on the pumping of an anhydrite, water,accelerator mixture into the packhole where the anhydrite slurry sets by hydrating to gypsum. Both crushed natural anhydrite and screened synthetic anhydrite are used in this system of gateroad support. Pack strength development is based on the following reaction :

$$\begin{array}{lcccl} \text{ANHYDRITE} & + & \text{WATER} & \longrightarrow & \text{DIHYDRITE (GYPSUM)} \\ CaSO_4 & + & 2H_2O & \longrightarrow & CaSO_4.2H_2O \end{array}$$

Since the rate of formation of dihydrite crystals is very slow it is necessary to add an accelerator to the hydration reaction. A good pack requires early strength. The presence of an accerlator dramatically increases the rate of gypsum crystal formation and provides excellent early strength qualities.

7.1 - NATURAL ANHYDRITE

Natural anhydrite packing system uses material that is crushed to a size distribution of 0-7 mm (National Coal Board,U.K, Recommendation) mixed with an accelerator of 1 parts by weight potassium sulphate (K_2SO_4) and 1.8 parts by weight ferrous sulphate ($FeSO_4.7H_2O$). The accelerator is added to the anhydrite in a quantity of 1% by weight and the anhydrite : water ratio is kept to 9 to 10% by weight. The anhydrite is pneumatically blown to the pack site where the water-accelerator mix is added before ejection. The anhydrite slurry will then hydrate to gypsum with accelerators providing the required reaction speed for rapid pack strength development.

Natural anhydrite can be pumped to distances of upto 1500 meters. Hydraulic transport, however, relies on a very fine material size disturbution with inter-mix of large granules. A large portion of the anhydrite must consist of very fine material in order that the anhydrite : water reaction occur rapidly and uniformly. This permits the formation of a finely granulated mixture which acts as a reconstituted mass in a similar way to aggregate in concrete and also provides the required size

7.2 - SYNTHETIC ANHYDRITE

Synthetic anhydrite is a by-product formed during hydrofluric acid production, phosphate fertilizer production or the gas desulpherization at the coal fired power plants. Because it is usually formed as an amorphous powder it cannot easily be adopted to penumatic transportation . A typical synthetic anhydrite size distribution consists of the majority of the total weight in dust with the largest granule not excedding 2-3 mm. Such a fine size distribution requires synthetic anhydrite to be slurried and then pumped into the packs.

The by weight ratio of the Anpck mix is :

	ANYHDRITE	ACCERLERATOR	WATER
NATURAL	100	2.8	10
SYNTHETIC	100	1	10

7.3 - TYPICAL INSTALLATIONS

7.3.1 - Alsteden System

This system of packing was first developed by Karl Briedtn and Co. of Germany and has remained the most extensively used packing system in Continental European Collieries. Natural anhydrite crushed to the required 0-7 mm size distribution is delivered to the colliery and stored in 50 ton silos on surface. from the silos the packing material can be easily fed into mine cars or monorail containers for transport underground. The anhydrite, once transported to the appropriate longwall panel, is then deposited into storage bunkers. An adjustable length, scraper-feed, chain conveyor feeds the anhydrite into the intake hopper of Brieden stower delivers the anhydrite, at a predetermined feed rate, into a compressed air stream of approximately 40 p.s.i. The intake hopper of the Brieden stower should be kept full at all times and must be specially coated with a hardened material to minimize corrosion. Special hardened basalt-lined pipes and fittings transport the pneumatically blown air-anhydrite mix to the pack area.

The accelerator and water and/or flyash are pre-mixed in a mixing tank located near the Brieden stowing machine and then pumped through 25 mm lines (line pressure=12.5 atm.)to within 5 m of the pack site. The accelerator salts dissolve in the water through the mixing tank. When the fluid level drops to 2/3 of the tank height, the tank is topped up with water and 1 bag (40 kg) of accelerator. At a location approximately 5 m from the pack hole the fluid accelerator stream and air-anhydrite stream are mixed using a special flanged annulus. This annulus incorporates a number of wetting nozzles that evenly mix the anhydrite, water and accelerator throughout the circumference of the annula. This special annulus is coupled to a 100 mm flexible discharge hose. The flexible hose allows the operator to direct the mix, the mixture being of a similar consistency to thick porridge, to any location within the pack site. This ensures a consistent, dense, monolithic pack. Because of compressed air requirements this system is limited to a range of around 1000 m of stowing pipe.

The anhydrite packs are lined with brattice cloth along the goaf and faces sides. Timbers are fitted between the arches to contain the slurry as the anhydrite pack builds up. Packing tends to be done after two face end cuts (4 ft.) and packs are usually 5-6 feet in width. Within 1 hour the pack is dry to touch and within two hours it can be considered solid.

This system can also be further simplified by adding the accelerator, in a dry state, to the anhydrite at the Breiden pneumatic mixing station. As long as a properly calibrated dosimeter is used to regulate the flow of the accelerator proper mixture ratios can be maintained. This simplification eliminates the need for an accelerator-water mixing tank and its associated manpower requirements. Besides the elimination of the water-accelerator mixing tank and the addition of a dosimeter and accelerator addition hopper on the Breiden stower the original layout and procedure remains the same. Man power requirements for the Alstadem system include 1 to 2 men to operate the Breiden machine and associated anhydrite hopper, 1 man to operate the water-accelerator mixing tank (if used), and one man to control the discharge of the anhydrite slurry into the pack hole. A portion of a face end shift may be required in preparing the anhydite pack screen or brattice curtain.

Pipe blockages associated with this system are minimal but abrasion is a serious problem. Pipes, hoppers and mine cars must be specially lined with a hardened protective liner. Replaceable basalt lines are used inthe pipes and pipe fittings. This system can output 8-20 dry tons/hour of natural anhydrite using compressed air quantities of 75 m^3/hour to 400 m^3/hour, depending on the distance the material is pneumatically blown.

7.4 - ADVANTAGES OF ANPAK

1: Simplicity of operation,

2: The cheapest of all pack types,

3: Displays high strength characteristics after a long curing time.

7.5 - DISADVANTAGES OF ANPAK

1: Low strength development within the first 2 hours of curing,

2: Corrosive in contact with metals underground,

3: Very brittle.

7.6 - COMPRESSIVE STRENGTH AND STRESS/STRAIN DEVELOPMENTS FOR ANPAK

This is shown in Figures 36 and 37 as typical values for standard mixture of ANPAK.

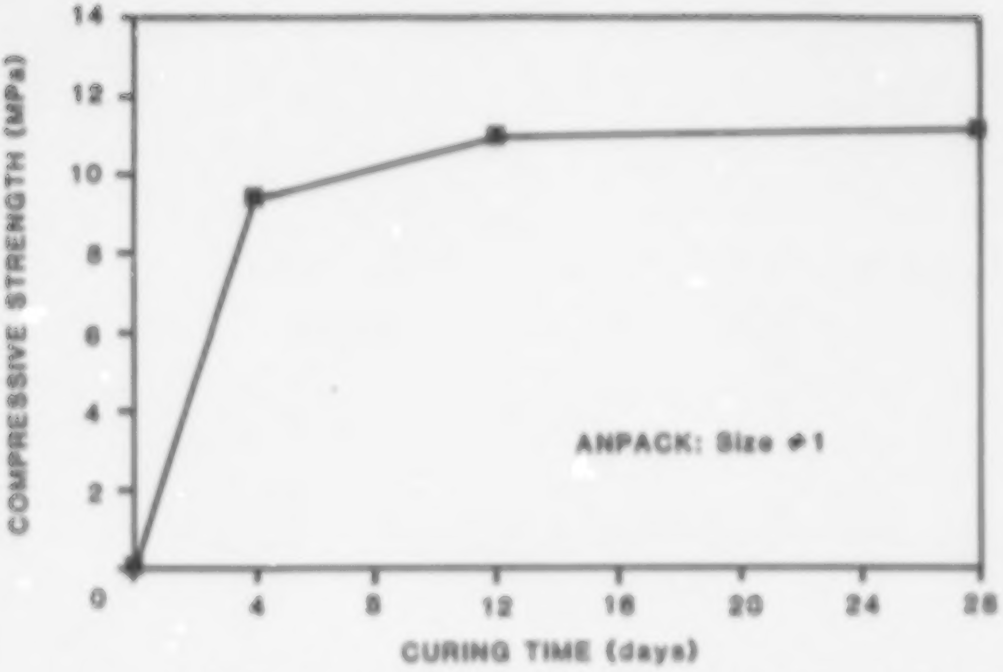


Figure 36 **STRENGTH DEVELOPMENT OF STANDARD ANPAK**

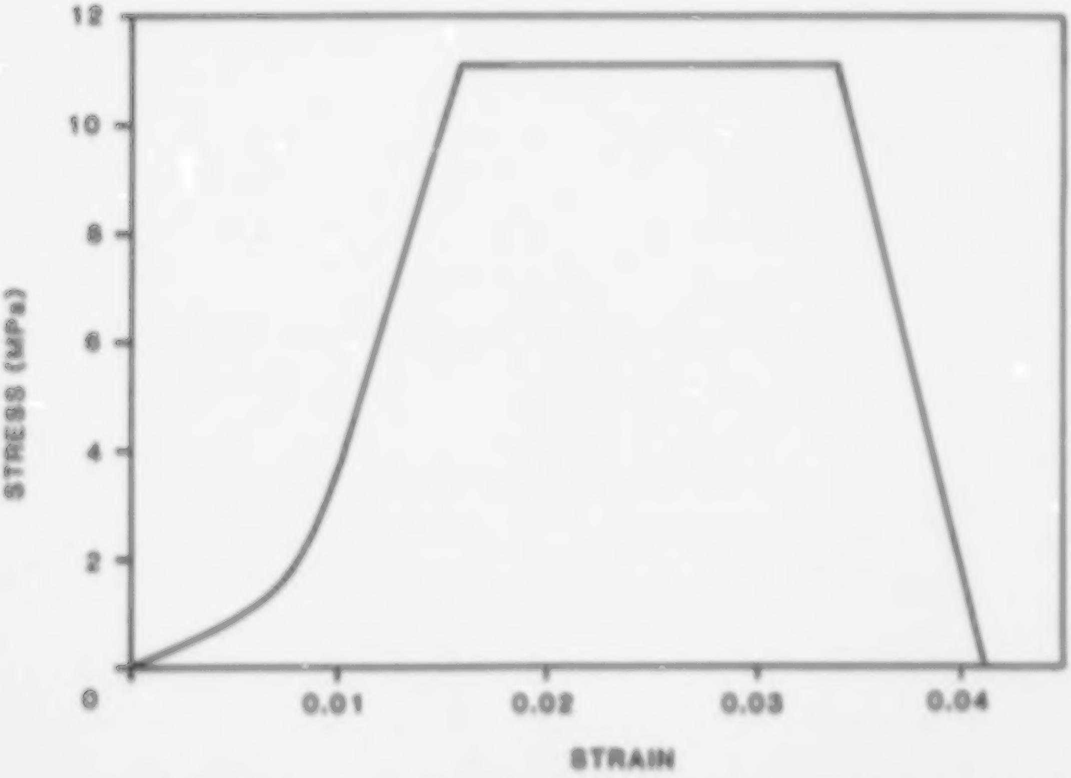


Figure 37 **TYPICAL STRESS/STRAIN CURVE FOR STANDARD ANPAK**

8 - FLYASH (FLASHPAK)

8.1 - PREPARATION

"Flashpak" is an MPP system, based on flyash and Ordinary portland cement. The flashpak is made from mixtures noted below:

Mixture I : flyash + accelerator + bentonite + water

The accelerator can be either Ferrous sulphate ($FeSO_4$) OR Aluminum sulphate ($Al_2(SO_4)_3$). The latter is more effective, but more costly. Usage of Ferrous sulphate decreases the early pack strength, but not enough to be a problem.

Mixture II : cement + water

This slurry has about 500 m pumping distance limit since it hardens in the pipe quickly.

The by weight ratio of the flashpak is :

	FLYASH	CEMENT	ACCELERATOR	BENTONITE	WATER
(%)	50.2	13.9	4.5	1	30.4

Both grouts are pumped to the face end where they are mixed in a mixing gun and sprayed into the prepared packhole. The volume of flyash slurry is approximately twice that of the cement slurry. The resulting mortar gels instantaneously, so the need for a waterproof containing bag is eliminated. With the cement pump adjusted to pump at half the rate of the flyash pump, the two slurries are then pumped simultaneously through one and quarter inch hoses into an ejector mixer device and allowed to spray into a container.

8.2 - COST EFFECTIVENESS

Both flyash and cement are locally avilable in Nova Scotia. Ferrous sulphate is only obtainable from Quebec, where it is a by-product of the manufacture of nails. Due to high cost of Ferrous sulphate delivered from Quebec ($225/tonne), it is more economical to import the Ferrous sulphate from the U.K, resulting in a savings of $30/tonne. Bentonite, from Wyoming, is avilable from the oil service industries in Halifax, Nova Scotia. Again, importing bentonite from the U.K would result in a savings of $66.5/tonne. The cost per tonne of each material is summarized below :

MATERIAL	LINGAN	IMPORTED
FLYASH	($30)	$8.2
CEMENT	($162)	$120
FERROUS SULPHATE	($225)	80 ($195)
BENTONITE	($330)	148.5 ($264)

NOTE: The values in ( ), are the delivered costs. The transportation cost from the U.K is $115/tonne.

This difference in costs per cubic metre between mixes based on local and imported materials would result in a saving of $ 46,000 on a 2500 m long face using a 2.1 m wide by 2.1 m high pack.

A typical face at Lingan Colliery, Sydney, Coalfield, Nova Scotia, advances at a rate of 18-20 m a week. Required strength/time characteristics of flashpak for a face advance of 18 m a week is shown below :

TIME	2 HR	1D	7D	14D	28D
STRENGTH(Mpa)	0.4	2.4	5.2	6.7	8-11

8.3 - ADVANTAGES OF FLASHPAK

1: Quick gelling properties. It needs no brattice cloth and weldmesh during the curing.

2: It reaches to a relatively high final strength after 15 days of curing.

3: Its cost is in the medium range of the monolithic packs.

4: The packing rate can be high and upto 22 m^3/hour.

8.4 - DISADVANTAGES OF FLASHPAK

1: Its initial strength development is low and can not act as a support during the first day of curing.

2: Since it gels quickly, each of the mixtures I and II (noted in section 8.1) must be flushed separately into the packhole, where they will be mixed together. Hence, a uniform mixture and quality control is difficult to achieve.

9 - ANHYDRITE AND FLYASH MIXTURE, ANFL

9.1 - SAMPLE SPECIFICATION AND PREPARATION

The flyash used was F ash, produced from bituminous coals and had a low lime content possessing little cementitious value by itself. This flyash has a minimum of 97% passing a 150 micro sieve and a minimum of 87% passing a 45 micro sieve. The anhydrite was a naturally occuring material (Fundy 2) which had been finely crushed. The grading and different sizes are given in Figure 38 . There are four different sizes of anhydrite which were selected to be tested in laboratory . The major particle size distribution used in testing was size no.1 . After receiving the anhydrite from Little Gypsum Company (sized 1-2 in.), the laboratory cone crusher was used to crush the anhydrite and obtain size no.1 .The coarse particles of size 1 were screened out to obtain size 2 . In order to get size 3 distribution , a ball mill was used to reduce the size of the material . Size 4 was the coarsest size distribution of anhydrite in mixes selected for testing . To facilitate the curing process of anhydrite , a small amount of accelerator was employed comprising 1.8 parts of Ferrous sulphate to one part Potassium sulphate

9.2 - UNCONFINED COMPRESSIVE TESTS

Three series of tests were carried out . In the first series, the standard Anpak was tested using locally available anhydrite from the Sydney area (Table 8). In the second series, varying proportions of anhydrite were mixed with flyash (Tables 9-16).

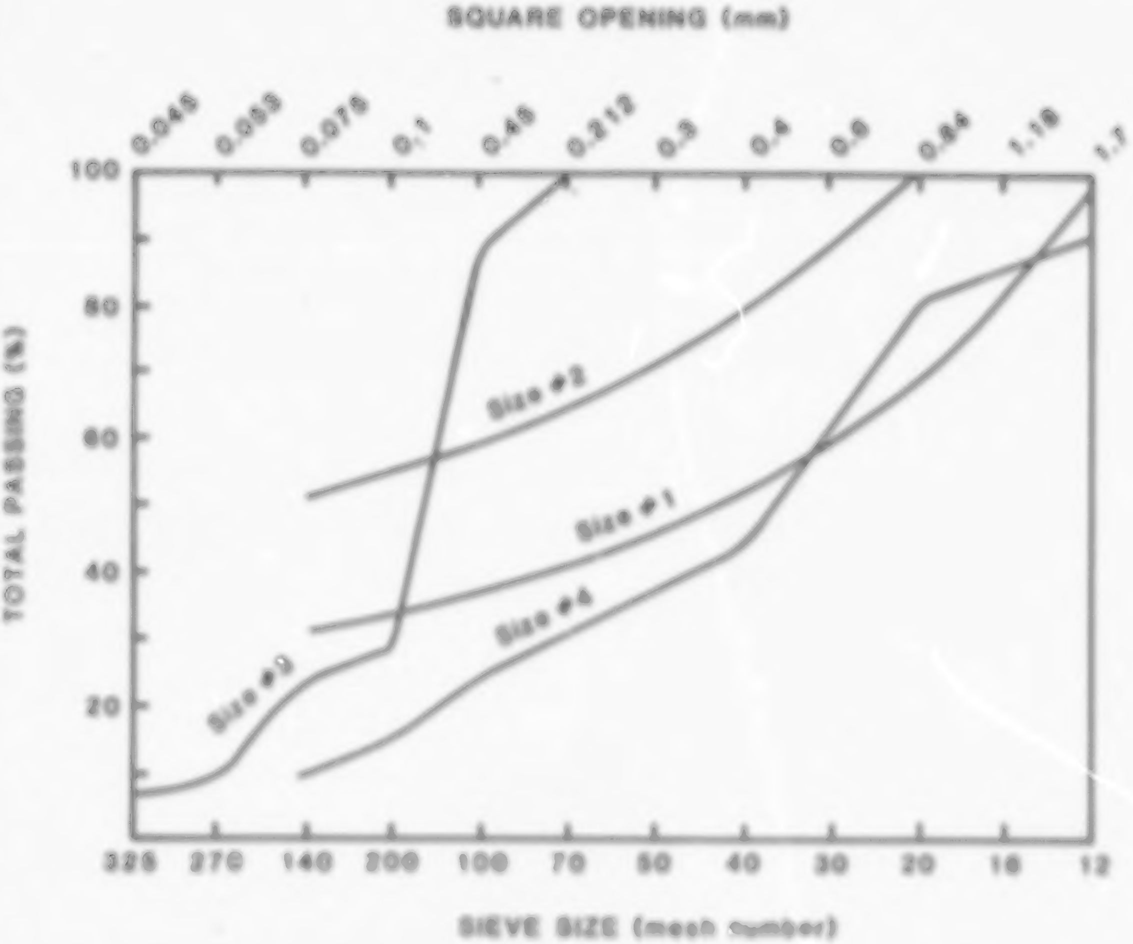


Figure 36 SIZE DISTRIBUTIONS OF NATURAL ANHYDRITE USED IN TESTING

SET A

THIS SET IS CALLES AN-PACK. HERE, SIZE 1 ANHYDRITE HAS BEEN USED.

1. ANHYDRITE = 100 PARTS
2. ACCELERATOR = 2.8 PARTS (1.8 of $FeSO_4$ and 1 P of K_2SO_4)
3. WATER = 10 PARTS

TYPE A (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	9.4	2424	1064
3	11	2368	1075
7	11.2	2307	1097
30	11.3	2292	1102

Table 8 STRENGTH CHARACTERISTICS OF STANDARD ANPACK

SET M

THIS IS A MIXTURE OF BOTH FLYASH AND ANHYDRITE. IN ALL THE FOLLOWING MIXTURES THE WATER CONTENT AND AMOUNT OF ACCELERATOR ARE KEPT CONSTANT.

1. FLYASH = 80 PARTS (66%)

2. ANHYDRITE = 20 PARTS (16.5%)

3. $FeSO_4$ = 0.77 PARTS (0.64%)

4. K_2SO_4 = 0.43 PARTS (0.33%)

5. WATER = 20 PARTS (16.5%)

TYPE M (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	0.4	2159	42
3	.61	2139	80
7	.54	2136	66
28	.72	2116	84

Table 9 STRENGTH CHARACTERISTICS OF ANFL-M SIZE #1

SET B

THIS IS THE SECOND MIXTURE OF FLYASH AND ANHYDRITE. THE ANHYDRITE USED HERE IS SIZE 1.

1. FLYASH = 60 PARTS (49.5%)
2. ANHYDRITE = 40 PARTS (33%)
3. $FeSO_4$ = 0.77 PARTS (0.64%)
4. K_2SO_4 = 0.43 PARTS (0.35%)
5. WATER = 20 PARTS (16.5%)

TYPE B (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	.51	2216	87
3	1.1	2205	137
7	1.43	2190	182
30	1.77	2180	241

Table 10 STRENGTH CHARACTERISTICS OF ANFL-B SIZE #1

SET C

THIS IS THE THIRD COMBINATION OF FLYASH AND ANHYDRITE. SIZE 1 ANHYDRITE HAS BEEN USED.

1. FLYASH = 40 PARTS (33%)
2. ANHYDRITE = 60 PARTS (49.5%)
3. $FeSO_4$ = 0.77 PARTS (0.64%)
4. K_2SO_4 = 0.43 PARTS (0.35%)
5. WATER = 20 PARTS (16.5%)

TYPE C (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	.64	2362	78
3	2.0	2317	311
7	2.6	2256	356
30	2.65	2246	403

Table 11 STRENGTH CHARACTERISTICS OF ANFL-C SIZE #1

SET T

IN THIS PART SET O, IS REPEATED WITH FINER SIZE OF ANHYDRITE, TO SEE THE EFFECT OF ANHYDRITE SIZE ON COMPRESSIVE STRENGTH.

1. FLYASH = 40 PARTS (33%)
2. ANHYDRITE = 60 PARTS (49.5%)
3. $FeSO_4$ = 0.77 PARTS (0.64%)
4. K_2SO_4 = 0.43 PARTS (0.35%)
5. WATER = 20 PARTS (16.5%)

TYPE T (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	2.54	2292	377
7	3.0	2277	779
30	6.45	2252	899

Table 12 **STRENGTH CHARACTERISTICS OF ANFL-O SIZE #2**

SET P

THIS IS THE FOURTH COMBINATION OF FLYASH AND ANHYDRITE. AGAIN, ANHYDRITE SIZE 1 HAS BEEN USED HERE.

1. FLYASH = 20 PARTS (16.5%)
2. ANHYDRITE = 80 PARTS (66%)
3. $FeSO_4$ = 0.77 PARTS (0.64%)
4. K_2SO_4 = 0.43 PARTS (0.35%)
5. WATER = 20 PARTS (16.5%)

TYPE P (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	1.62	2358	293
3	3.3	2348	579
7	5.13	2326	416
30	5.63	2312	569

Table 13 **STRENGTH CHARACTERISTICS OF ANFL-P SIZE #1**

SET S

HERE, SET P HAS BEEN REPEATED WITH FINER SIZE ANHYDRITE. EVERY OTHER PARAMATERS HAS BEEN KEPT THE SAME AS SET P.

1. FLYASH = 20 PARTS (16.5%)
2. ANHYDRITE = 80 PARTS (66%)
3. $FeSo_4$ = 0.77 PARTS (0.64%)
4. K_2SO_4 = 0.43 PARTS (0.35%)
5. WATER = 20 PARTS (16.5%)

TYPE S (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	3.47	2309	599
7	7.2	2292	816
30	11.1	2277	1193

Table 14 **STRENGTH CHARACTERISTICS OF ANFL-P SIZE #2**

SET R

SET R IS THE SAME AS SET S WITH ADDITION OF 5 PARTS OF LIME.

1. FLYASH = 20 PARTS (15.7%)
2. ANHYDRITE = 80 PARTS (63[illegible]
3. LIME = 5 PARTS (3.9%)
4. $FeSO_4$ = 0.77 PARTS (0.61%)
5. K_2SO_4 = 0.43 PARTS (0.34%)

TYPE R (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	2.85	2316	345
7	10.2	2293	849
30	11.1	2269	983

Table 15 **STRENGTH CHARACTERISTICS OF ANFL-P (size #2 + 5p lime)**

SET Q

SET Q IS THE SAME AS SET S WITH ADDITION OF 10 PARTS OF LIME.

1. FLYASH = 20 PARTS (15%)
2. ANHYDRITE = 80 PARTS (60%)
3. LIME = 10 PARTS (7.5%)
4. $FeSo_4$ = 0.77 PARTS (0.58%)
5. K_2SO_4 = 0.43 PARTS (0.32%)

TYPE Q (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
1	1.87	2241	419
7	5.5	2204	623
30	11.1	2198	1122

Table 16 **STRENGTH CHARACTERISTICS OF ANFL-P (size #2 + 10p lime)**

The third series , was the evaluation of the washplant tailings as fill material (Tables 17 and 18).

The uniaxial compressive strength tests were conducted , as described previously , on 50 mm diameter x 100 mm length cylinders at a constant displacement rate of 1mm per minute . A minimum of three uniaxial tests were carried out for each parameter set and the test procedures accord with the standard of the I.S.R.M.[7].

The materials were dry-mixed until a uniform appearance was achieved . Meanwhile the accelerator , with a weight equal to 1% of the total weight of anhydrite and/or flyash , was dissolved in a measured quantity of water . This solution was then mixed with the dry constituents.

9.3 - TEST RESULTS

Table 19 shows the constituents, density , modulus of elasticity , cost and compressive strength of each combination that was tested. This table also shows the potential savings that could be achieved using the local resources . The average cost per m^3 for the locally available pack (ANFL) is around $35 plus grinding costs and for commercially available packs , such as Astrapak , the cost is around $110 .

In all the different mixes of anhydrite and flyash ,the amount ,by weights, of water and accelerator were kept constant . Figures 39 and 40 show that the finer anhydrite (size 2) has increased the compressive strength by 140% after 28 days of curing. The figure also states that the early strength, 1 day ,

SET SR

THIS SET IS STANDARD WARBRET SYSTEM. FILLER IS -2 CM R.O.M. TAILING.

1. CEMENT = 19%
2. ACCELERATOR ($FeSO_4$) = 2 %
3. BENTONITE = 1%
4. TAILING FROM WASHPLANT = 48%
5. WATER = 30%

TYPE SR (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
3	0.65	1678	139
7	2.01	1543	218
30	3.05	1495	295

Table 17 **STRENGTH CHARACTERISTICS OF PUMP PACK #1 SYSTEM**

SET 2S

THIS SET IS STANDARD WARBERT SYSTEM PLUS 50% EXCESS CEMENT AND 50% EXCESS ACCELERATOR.

1. CEMENT = 28.5%
2. ACCELERATOR ($FeSO_4$) = 3%
3. BENTONITE = 1%
4. TAILING FROM WASHPLANT = 48%
5. WATER = 30%

TYPE 2S (DAYS)	COMP.STRENGTH MPa	DENSITY kg/m^3	MOD.ELAST MPa
3	.27	1914	32
7	4.6	1674	693
30	3.8	1625	517

Table 18 **STRENGTH CHARACTERISTICS OF PUMP PACK #2 SYSTEM**

System	Materials (%)	Quantity of Materials (kg)	Individual and Total costs	Curing Time (days)	c Mpa	Density kg/m3	Modulus of Elasticity
Anpack (size 1)	Anhydrite = 89	2157	32.0	1	9.4	2424	1064
	$FeSO_4$ = 1.6	39	7.6	3	11	2368	1075
	K_2SO_4 = 0.9	22	4.6	7	11.2	2307	1097
	Water = 8.9			28	11.2	2292	1102
			---- $44.2				
ANFL-M (size 1)	Flyash = 66	1425	18.5	1	0.4	2159	42
	Anhydrite = 16.5	356	5.3	3	.61	2139	88
	$FeSO_4$ = 0.64	13.8	2.7	7	.54	2134	46
	K_2SO_4 = 0.35	7.6	1.6	28	.72	2114	84
	Water = 16.5						
			---- $28.1				
ANFL-N (size 1)	Flyash = 49.5	1097	14.3	1	.51	2216	87
	Anhydrite = 33	731	11	3	1.1	2205	137
	$FeSO_4$ = 0.64	14.2	2.8	7	1.43	2190	182
	K_2SO_4 = 0.35	7.8	1.6	28	1.77	2180	241
	Water = 16.5						
			---- $29.7				
ANFL-O (size 1)	Flyash = 33	773	10.0	1	0.64	2342	78
	Anhydrite = 49.5	1160	17.4	3	2.0	2317	311
	$FeSO_4$ = 0.64	15	2.9	7	2.6	2256	356
	K_2SO_4 = 0.35	8.2	1.7	28	2.65	2246	403
	Water = 16.5						
			---- $32.0				
ANFL-P (size 1)	Flyash = 16.5	389	5.1	1	1.62	2358	293
	Anhydrite = 66	1556	23.3	3	3.3	2348	579
	$FeSO_4$ = 0.64	15.1	2.9	7	5.13	2326	716
	K_2SO_4 = 0.35	8.2	1.7	28	5.63	2312	792
	Water = 16.5						
			---- $33.0				

Table 19 STRENGTH AND COST COMPARISON BETWEEN ALL DIFFERENT COMBINATION

System	Materials (%)	Quantity of Materials (kg)	Individual and Total costs	Curing Time (Days)	c Mpa	Density kg/m3	Modulus of Elasticity
	Flyash = 33	756	9.8				
ANFL-O	Anhydrite = 49.5	1134	17.0	1	2.54	2292	377
(size 2)	$FeSO_4$ = 0.64	14.7	2.9	7	3.0	2277	779
	K_2SO_4 = 0.35	8.0	1.7	28	6.45	2252	899
	Water = 16.5						

			\$31.4				
	Flyash = 16.5	381	4.95				
ANFL-P	Anhydrite = 66	1524	22.9	1	3.47	2309	599
(size 2)	$FeSO_4$ = 0.64	14.8	2.9	7	7.2	2292	816
	K_2SO_4 = 0.35	8.1	1.7	28	11.1	2272	1193
	Water = 16.5						

			\$32.4				
1 ANFL-P	Flyash = 15.7	364	4.7				
(size 2)	Anhydrite = 63	1460	22	1	2.85	2316	345
+	Lime = 3.9	90.3	3.9	7	10.2	2293	849
5 parts	$FeSO_4$ = 0.61	14.1	3.0	28	11.1	2269	983
Lime	K_2SO_4 = 0.35	8.1	1.6				
	Water = 16.5						

			\$35.2				
	Flyash = 15	336	4.4				
ANFL-P	Anhydrite = 60	1345	20.2	1	1.87	2241	419
(size 2)	Lime = 7.5	168	7.2	7	5.5	2204	623
+	$FeSO_4$ = 0.58	13	2.5	28	11.1	2198	1122
10 parts	K_2SO_4 = 0.32	7.2	1.4				
Lime	Water = 16.5						

			\$35.7				

Table 19 **STRENGTH AND COST COMPARISON BETWEEN ALL DIFFERENT COMBINATION (CONT'D)**

System	Materials (%)	Quantity of Materials (kg)	Individual and Total costs	Curing Time (days)	c Mpa	Density kg/m3	Modulus of Elasticity
	Flyash = 16.5	386	5.0				
	Anhydrite = 66	1544	23.16	3	3.9	2340	812
ANFL-P	$FeSO_4$ = 0.64	15	2.9	7	7.5	2301	1422
(size 3)	K_2SO_4 = 0.35	8.2	1.7	28	15.3	2289	1687
	Water = 16.5						

			$32.8				
	Flyash = 16.5	389	5.0				
	Anhydrite = 66	1554	23.3	3	0.57	2355	121
ANFL-P	$FeSO_4$ = 0.64	15.1	2.9	7	1.66	2187	180
(Size 4)	K_2SO_4 = 0.35	8.2	1.7	28	3.3	2177	502
	Water = 16.5						

			$32.9				
	O.P.C. = 19	319	40.5				
Pump	$FeSO_4$ = 2	34	6.63	3	0.65	1678	139
Pack 1	Bentomite = 1	17	4.5	7	2.01	1543	218
	Tailing = 48	805	5.6	28	3.05	1495	295
	Water = 30	503					

			$57.3				
	O.P.C. = 28.5	545	69.2				
Pump	$FeSO_4$ = 3	57	11.1	3	.27	1914	32
Pack 2	Bentonite = 1	19.1	5.0	7	4.6	1674	693
	Tailing = 48	918.7	6.4	28	3.8	1625	517
	Water = 30						

			$91.7				

Table 19 STRENGTH AND COST COMPARISON BETWEEN ALL DIFFERENT COMBINATION (CONT'D)

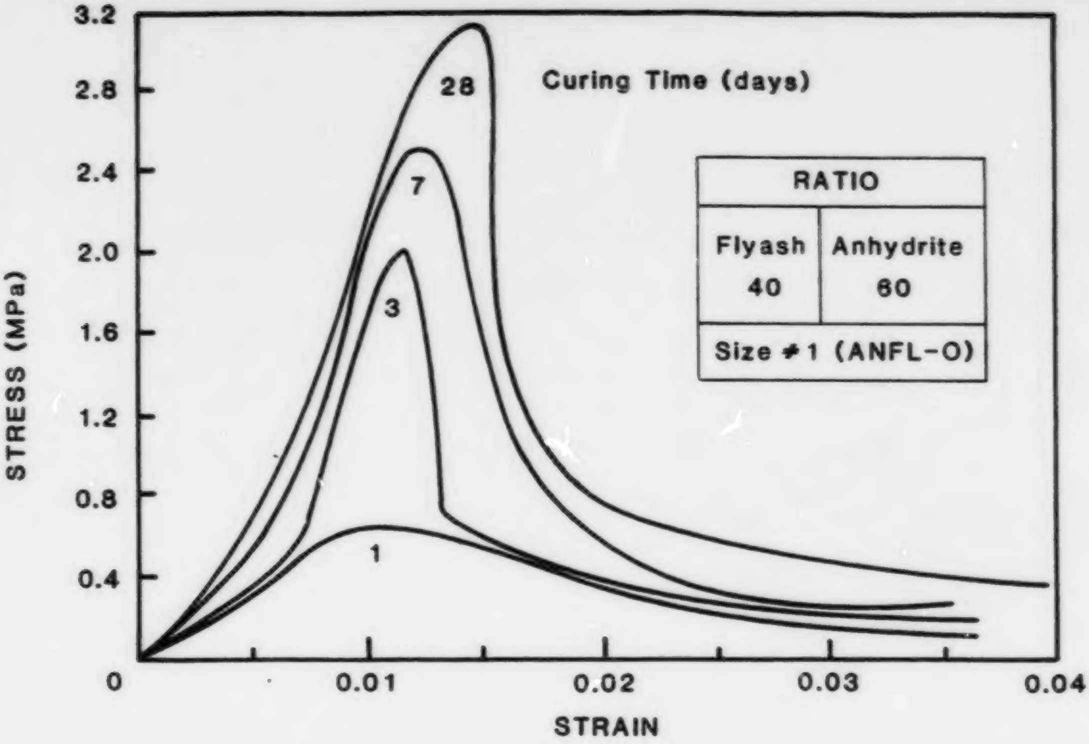


Figure 39 **STRESS/STRAIN CURVES FOR ANFL-O SIZE #1**

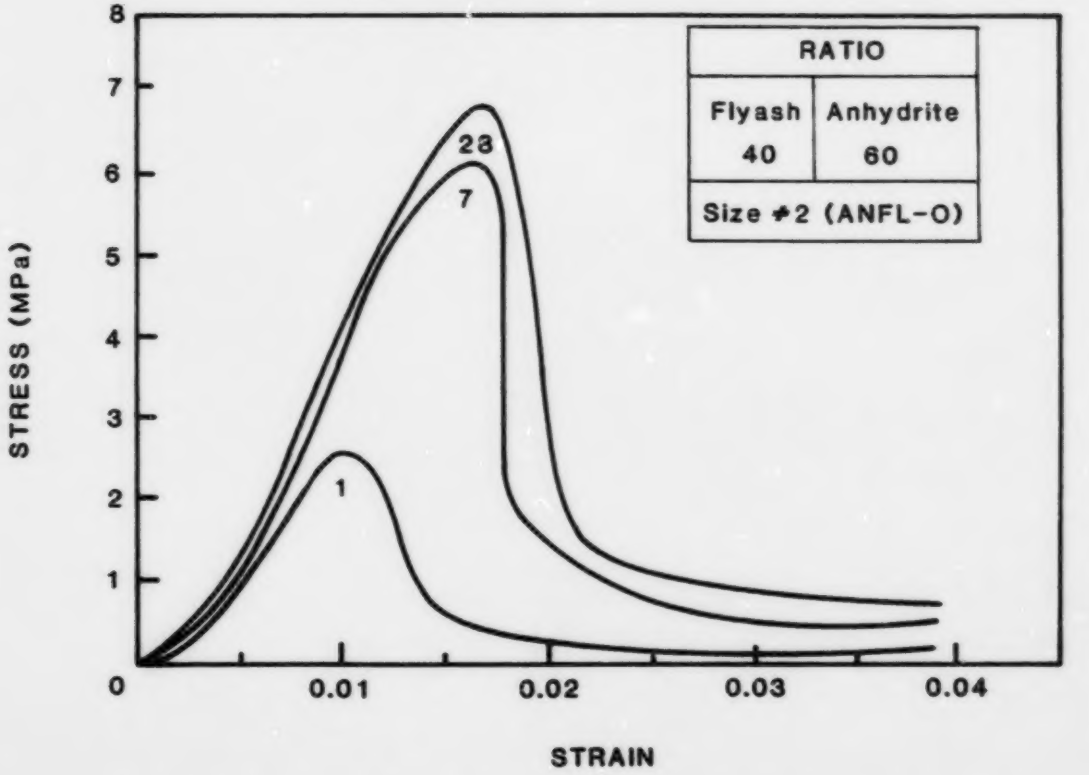


Figure 40 **STRESS/STRAIN CURVES FOR ANFL-O SIZE #2**

is increased by almost 300% . The more anhydrite used in the sample, the more pronounced the effect of size distribution on compressive strength (Figures 39-47). The compressive strengths and curing times for all the flyash/anhydrite mixes are shown in Figure 48. It can be seen that the addition of up to 40% anhydrite by weight to flyash has little effect upon the compressive strength. The data for flyash/anhydrite mixes show that between 60% and 80% of the 28-day strength is gained after 7 days.

Type F flyash, locally available in the Sydney area, has a low lime content and addition of lime to the mixes was investigated (Figure 49). This Figure shows that the addition of lime does not result in any increase in compressive strength for 28-day curing.This is perhaps due to small amount of flyash, 16.5% , used in the specific mix under study.

The effect of size distribution of anhydrite on compressive strength of 20:80 flyash to anhydrite ratio mix was studied and presented in Figure 45. Reducing the size of anhydrite increases the compressive strength of the mix and the rate of increase varies for different curing times. In selecting one of the available four sizes for use underground, an economical evaluation of the capital costs involved, especially in grinding and transportation costs, should be carried out. If pneumatic transportation is chosen the size distribution no.3 is the most suitable one. In case of hydraulic transportation all four sizes are possible alternatives. In the third series of tests, washplant tailings,

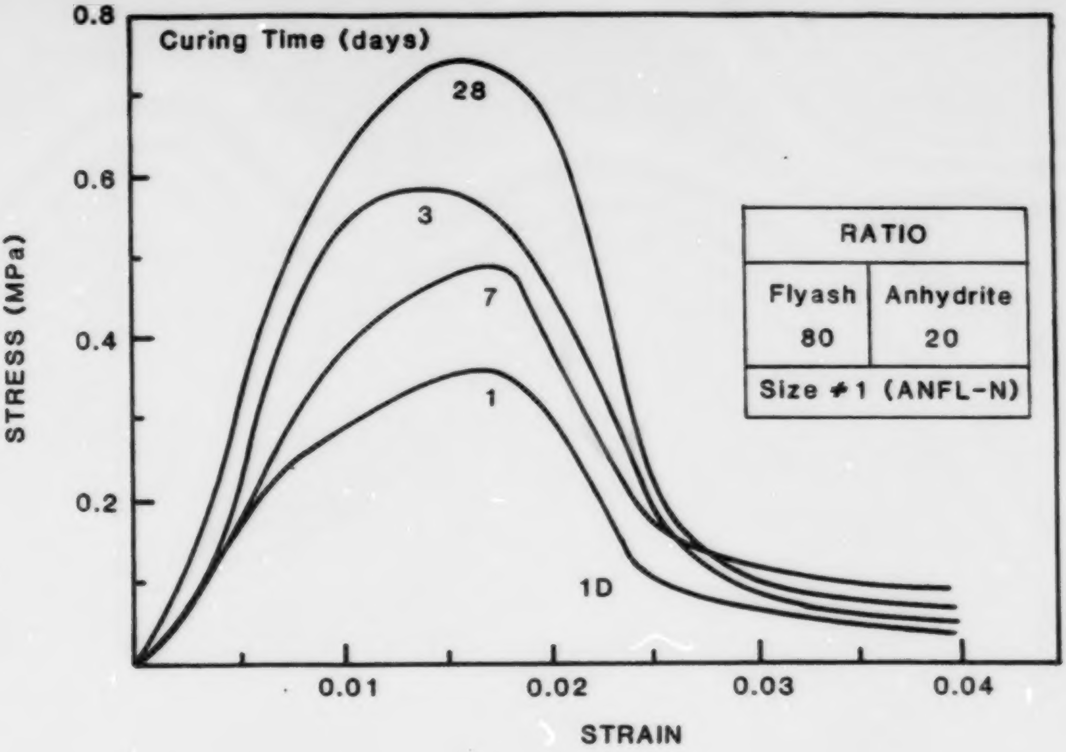


Figure 41 **STRESS/STRAIN CURVES FOR ANFL-N (size #1)**

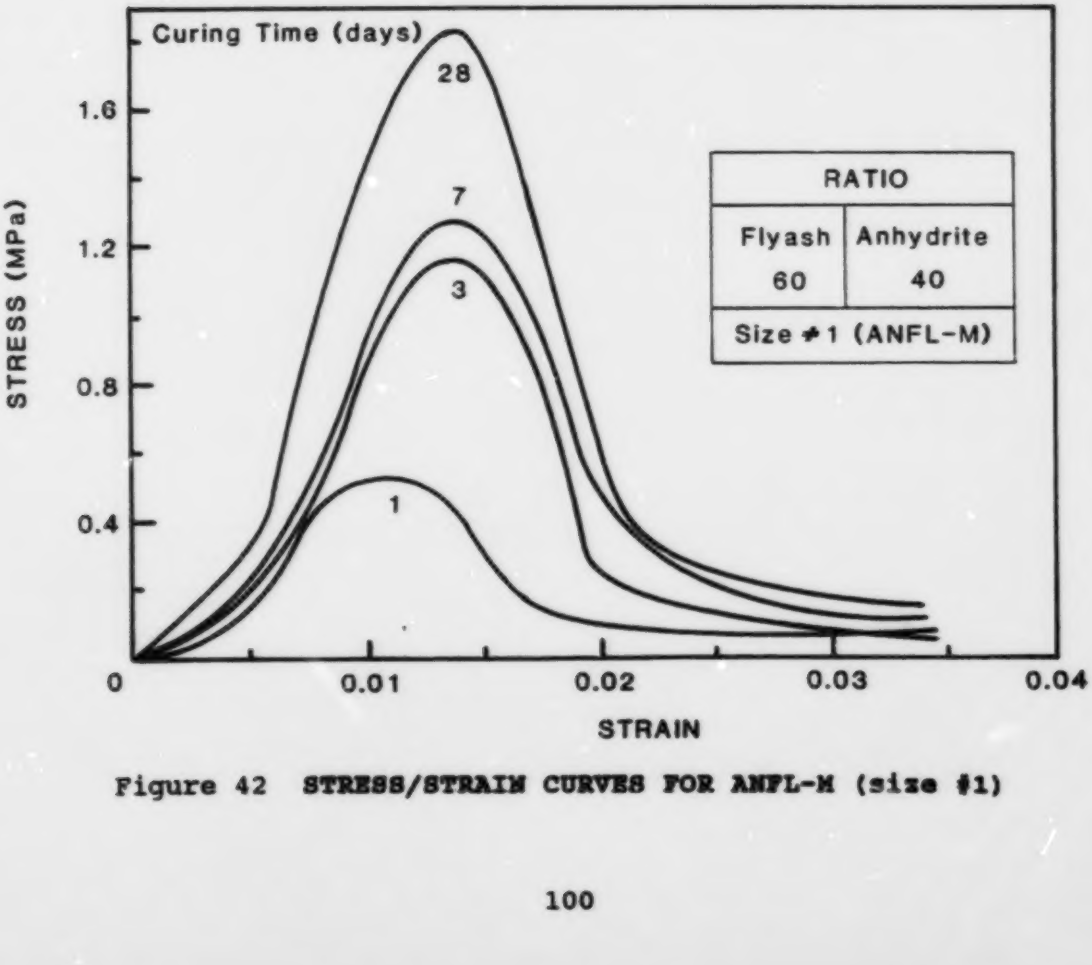


Figure 42 **STRESS/STRAIN CURVES FOR ANFL-M (size #1)**

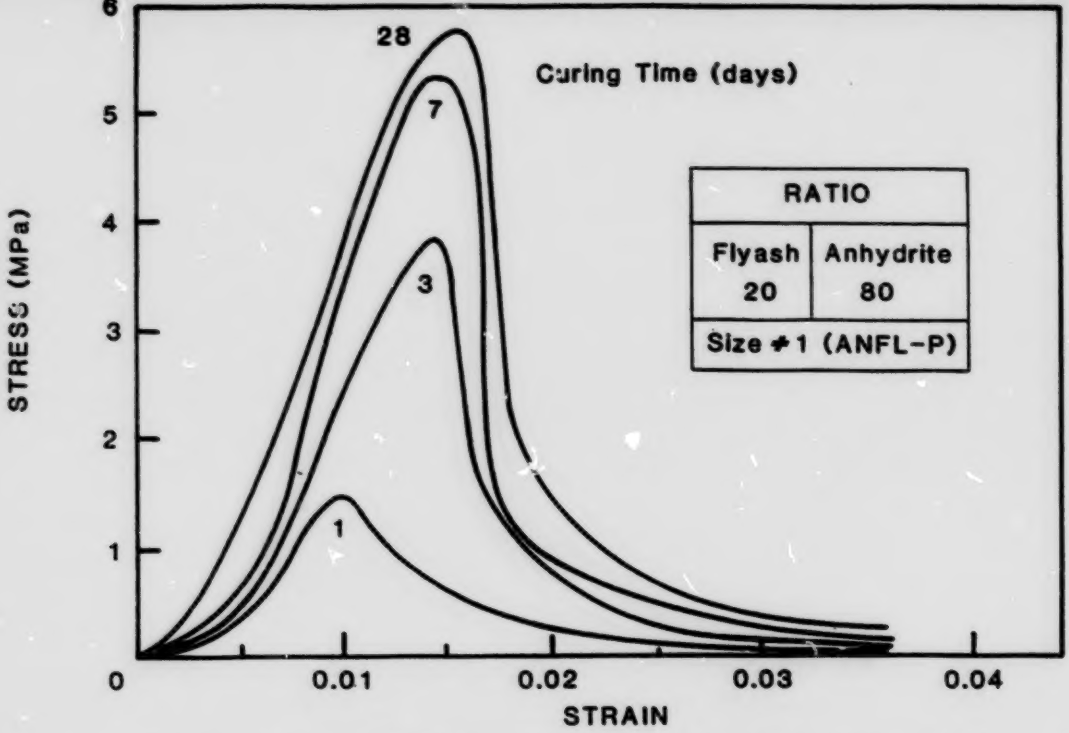


Figure 43 **STRESS/STRAIN CURVES FOR ANFL-P SIZE #1**

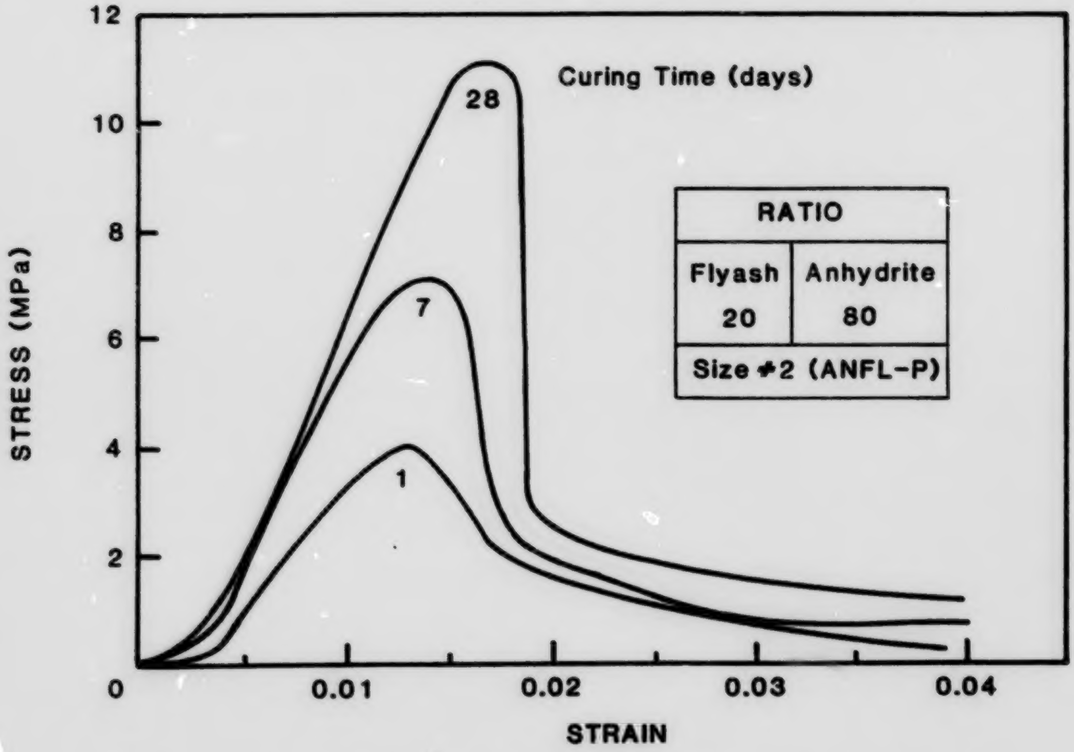


Figure 44 **STRESS/STRAIN CURVES FOR ANFL-P SIZE #2**

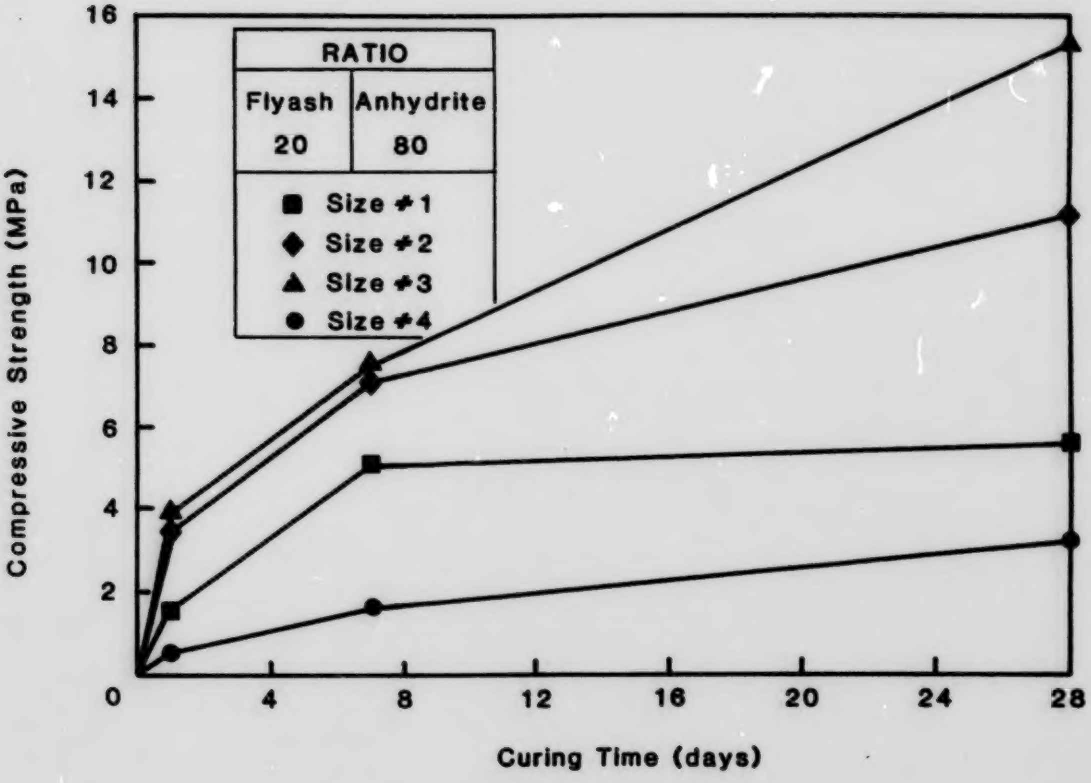


Figure 45 **STRENGTH DEVELOPMENT OF ANFL-P (all four sizes)**

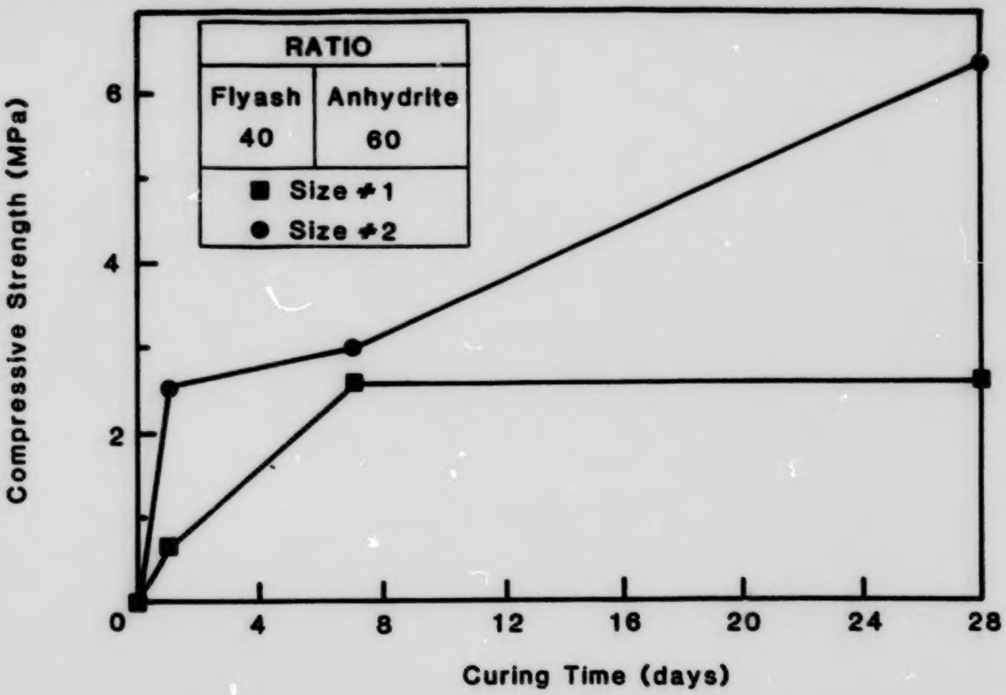


Figure 46 **STRENGTH DEVELOPMENTS OF AMFL-O (sizes #1 and #2)**

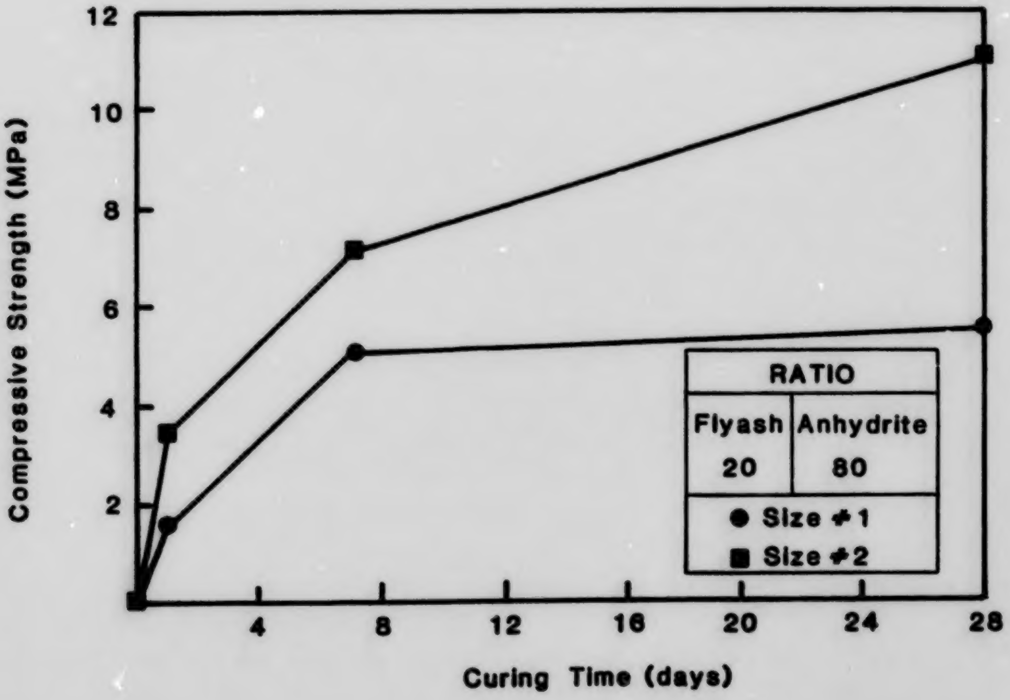


Figure 47 **STRENGTH DEVELOPMENT OF AMFL-P (sizes #1 and #2)**

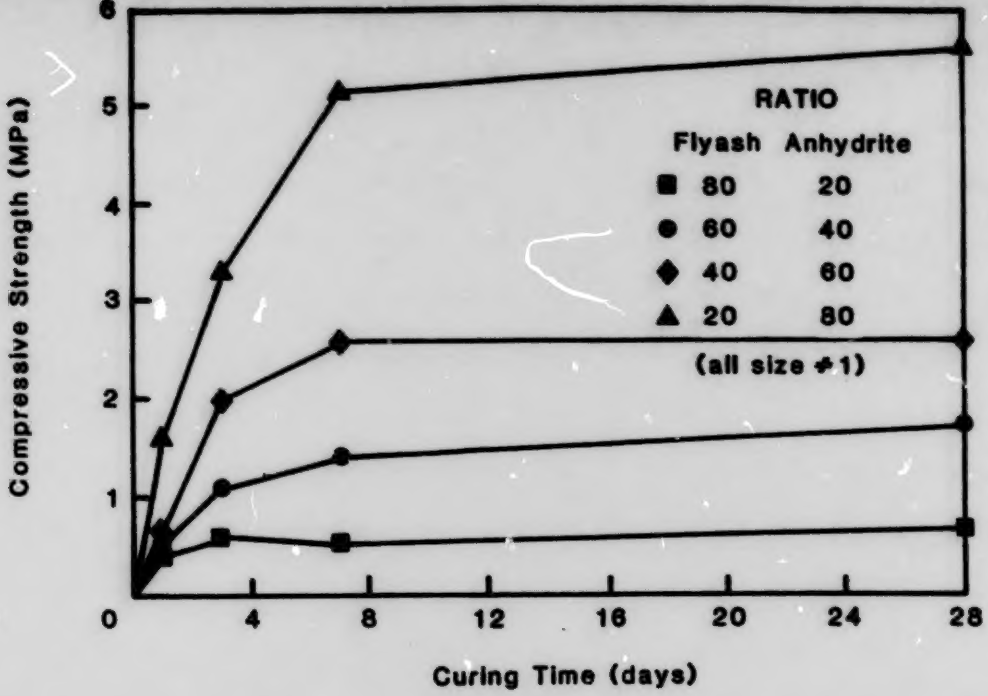


Figure 48 STRENGTH DEVELOPMENTS OF ALL ANFL COMBINATIONS

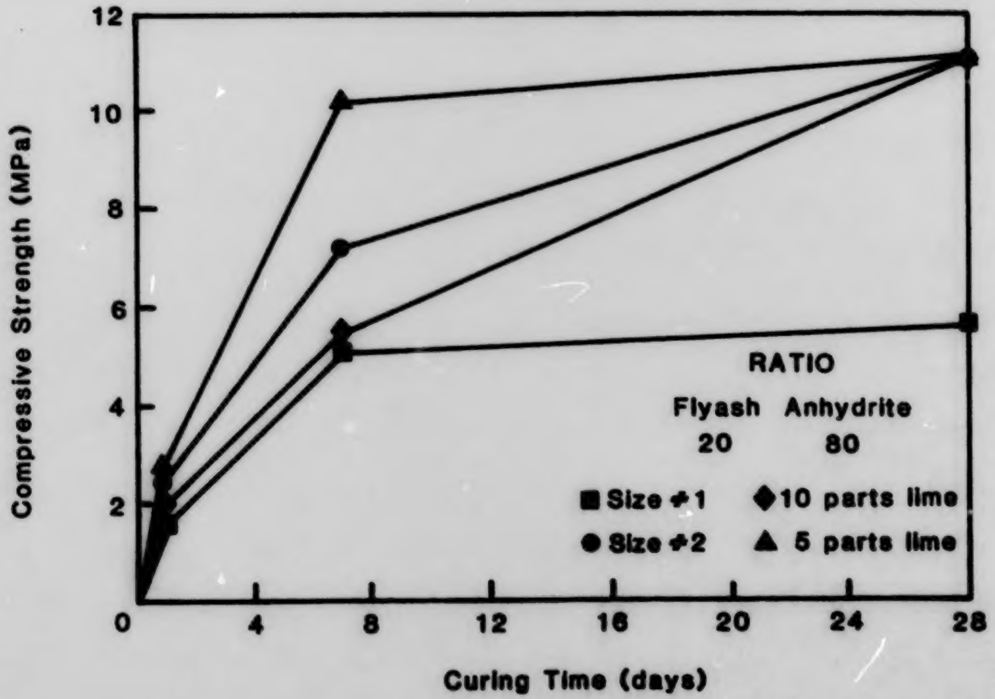


Figure 49 STRENGTH DEVELOPMENTS OF ANFL-P + LIME

were used as fill material in a standard Warbert system. The quantities of materials used in this system are reported in Table 19. Figure 50 shows that using tailing-based pack gives an acceptable longterm strength, but has a very low early strength. In order to improve the early strength of the pack, 50% excess cement and accelerator were added to the standard mix. The 7-day compressive strength of the new mix increased by 130% compared to the one of the standard Warbert system (Figure 51). However, the new mix started to deteriorate around 7 days of curing and this is perhaps due to high amount of accelerator in the mix. The 28-day compressive strength of the mix decreased by 20% compared to the 7-day strength. Typical stress/strain curves for most of the combinations tested are presented in Apendix A.

9.4 - DISCUSSION OF THE RESULTS

a: The water percentage by weight in all the ANFL combinations was kept constant at 16.5% . For anhydrite size distributions 1, 2 and 4, this was the minimum water content needed for hydraulic transportation . More water should be added when anhydrite size 3 (the finest size) is used and this is due to increased surface area of contact. If pneumatic transportation is chosen , water content could drop up to 9% by weight. At ratios less than 9% incomplete hydration occurs which reduces the strength of the mixture. At ratios greater than 9% , the presence of water leads to an excessive reduction in compressive strength, probably due to an increased void space in the mixture.

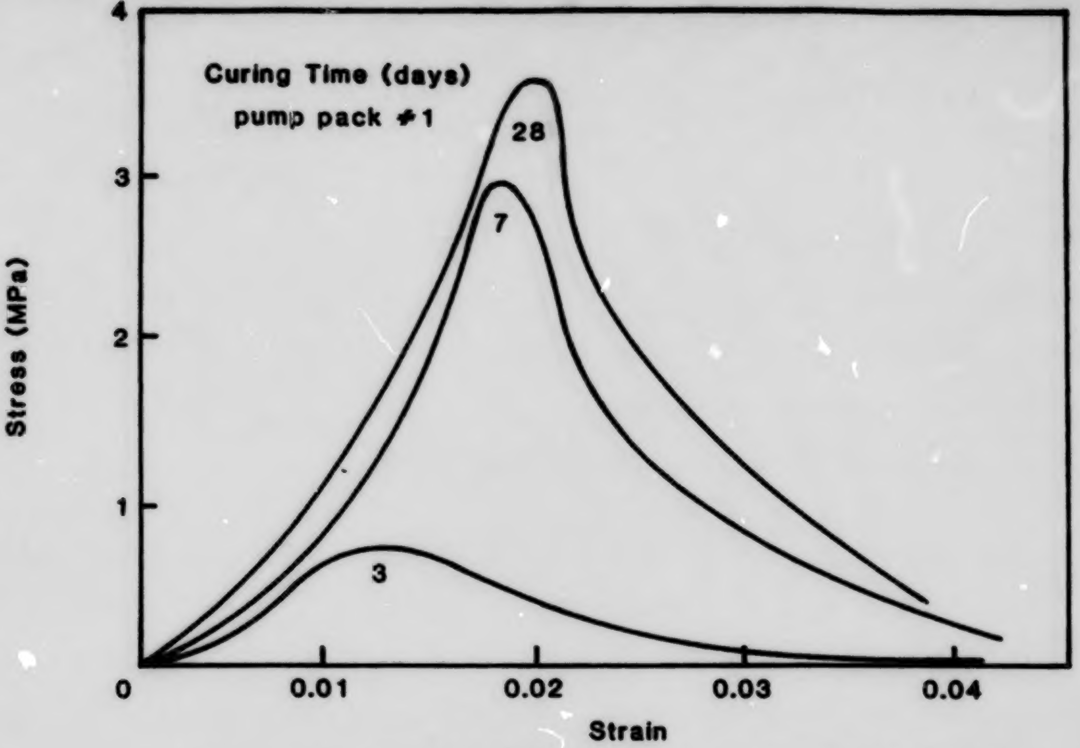


Figure 50 STRESS/STRAIN CURVES FOR PUMP PACK #1

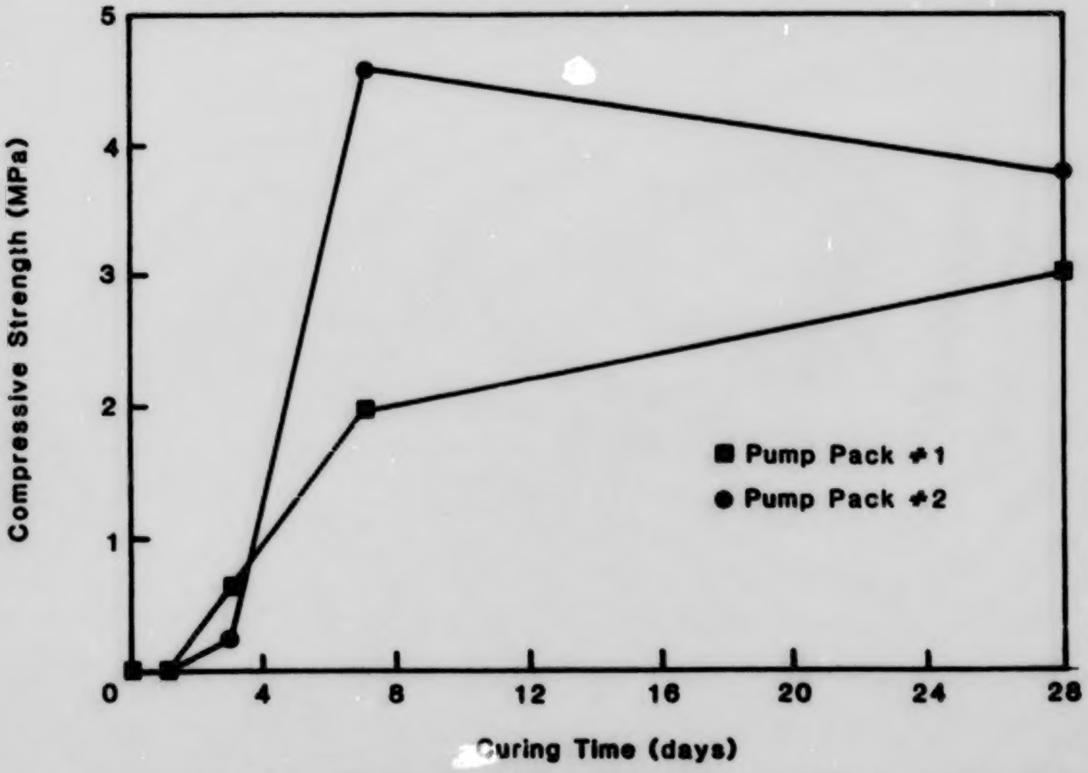


Figure 51 STRENGTH DEVELOPMENTS OF #1 AND #2 PUMP PACKS

b: The maximum value of 3 day compressive strength occurs at progressively higher water/anfl ratios as the grain size is reduced. This is probably due to the increased surface area of the finer grains providing more surface area and better reactivity of the material with water.

Following suggestions can be made from Table 19:

a: All ANFL-P combinations are feasable as a packing system in the Sydney Coalfield.

b: ANFL-P SIZE 3 is suitable for pneumatic transportation. Sizes 1 and 2 are the most suitable combinations for hydraulic transportation.

c: ANFL-M,N and ANFL-O (size 1) have a very low compressive strength and are not applicable for use underground.

d: Addition of lime to ANFL-P combination did not effect the longterm strength.

e: Pump pack 1 did not offer adequate early strength and more investigation into the different types and amounts of accelerator should be carried out.

f: Addition of 50% excess cement and accerelator to the standard mix (pump pack 1) did not result in increased strength.

g: Figure 46 states that the 1-day compressive strength of the ANFL-O combination has increased by almost 300% when size 1 anhydrite is replaced with size 2. There is also a sharp increase in compressive strength after 7-days of curing.

h: Standard ANPACK system is very brittle and could create floor heave problems in weak strata.

i: All ANFL combinations,except ANFL-P size 4, are easily flowable and should have excellent bearing contact with the roof and floor. This is a desirable characteristic since the pack will accept load equally over the entire cross-section.

j: ANFL-P sizes 2 and 3 are the most suitable systems for use in the Sydney Coalfield. They provide the required strength with the minimum cost. These combinations provide very high early pack resistance and low convergence rates enabling better strata bridging over the roadway and immediate load acceptance (resistance to downward roof movement and upward floor movement is offered at a very early stage).

10 - ANHYDRITE AND MINE TAILING MIX (ANTAIL)

10.1 - Sample preparation

The ANTAIL was made from two mixtures given as follows:

Mixture 1 - comprising Anhudrite + Mine Tailing + Water. The mixing time was kept at half a minute.

Mixture 2 - consisting of Ferro sulphate accelerator ($FeSO_4$, $7H_2O$) + water. The mixing time was kept at half a minute.

The mixtures 1 and 2 are then added together and mixed for an additional half a minute.

Parts by weight of each of the above constituents in the final mixture is given in Table 20. All possible variations of the constituents and their quantities within the range noted in Table 20 was made.

The final mixture was then pored into PVC cylindrical molds. The mold dimensions was 150 mm long x 50 mm internal diamteter. After the relevant curing time, both ends of the hardened sample was cut to eliminate the end effects during poring and curing. The final sample length was made 100 mm. This gave a length to diameter ratio of 2:1 to comply with the I.S.R.M. [7].

Table 20 - Parts by weight of the constituents for the various ANTAIL samples tested (all possible variation of the constitutents and their quantities was made).

Anhydrite sizes (mesh) :	-20 to +50		-50 to +100		-100		-200
Anhydrite/Tailing ratio :	25/75		50/50		75/25		100/0
Accelerator :		2.8		3.6			
Water (at 19-20°C) :		25		30		35	
Curing time (days) :	3		7		14		28

10.2 - UNCONFINED COMPRESSIVE STRENGTH (U.C.S.) AND THE STRESS/STRAIN TEST RESULTS

This test was conducted on all the samples at a constant displacement rate of 1 mm per minute.

Average of the test results including the density and strain reading is given in Table 21. Representative curves of the U.C.S. vs. curing time for various samples is given in Figures 52 to 71. The Stress vs. Strain curves are included in the Appendix (A15 to A192). The test results show:

(a) Different mixtures have shown improvement in the U.C.S. with decrease in the mesh size of the anhydrite. The maximum U.C.S.-value resulted for -200 mesh anhydrites in the mix. (Fig. 52).

(b) Additional accelerator [$FeSO_4$, $7H_2O$], beyound the optimum of 2.8 parts by total weight of the mix does not improve the U.C.S. of the ANTAIL.

(c) Decreasing the ratio of anhydrate:tailing resulted in decrease of the U.C.S. This was especially pronounced after 28 days of curing (Figs. 52-29).

(d) The U.C.S. was directly proportional to the curing time, during 28 days of curing. However, more test should be conducted to see how non-linear is the hardening rate of the ANTAIL.

Table 21 - UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS FOR THE ANTAIL.

Serial No.	Mesh Size	Curing Days	Anhydrite (Parts)	Tailings (Parts)	Water (Parts)	FeSO4 (Parts)	Density (gr. cu.cm.)	U.C.S. (MPa)	Strain (%)
1	-200	3	50	50	25	2.8	2.030	3.728	0.4
2	-200	3	25	75	25	2.8	2.151	1.570	0.2
3	-200	3	50	50	30	2.8	2.033	2.943	0.3
4	-200	3	25	75	30	2.8	1.962	0.294	0.3
5	-200	3	50	50	35	2.8	2.013	2.158	0.5
6	-200	3	25	75	35	2.8	2.091	0.392	0.2
7	-200	3	50	50	25	3.6	2.031	3.139	0.3
8	-200	3	25	75	25	3.6	2.093	1.472	0.3
9	-200	3	50	50	30	3.6	2.028	2.747	0.2
10	-50	7	50	50	32	2.8	1.924	2.256	0.3
11	-50	7	25	75	32	2.8	1.898	0.491	0.1
12	-50	7	50	50	37	2.8	1.834	0.589	0.4
13	-50	7	25	75	37	2.8	1.933	0.294	0.4
14	-50	7	50	50	42	2.8	1.835	0.491	0.1
15	-50	7	50	50	32	3.6	1.938	1.472	0.6
16	-50	7	50	50	37	3.6	1.898	1.177	0.5
17	-50	7	25	75	37	3.6	1.958	0.687	0.4
18	-50	7	50	50	42	3.6	1.842	0.491	0.4
19	-100	7	100	0	25	2.8	2.095	16.579	1.3
20	-100	7	75	25	25	2.8	2.066	16.187	0.9
21	-100	7	50	50	25	2.8	2.046	11.380	0.6
22	-100	7	25	75	25	2.8	2.035	2.060	0.7
23	-100	7	100	0	30	2.8	2.011	16.481	1.2
24	-100	7	75	25	30	2.8	1.963	15.696	0.9
25	-100	7	50	50	30	2.8	1.992	9.516	0.7
26	-100	7	25	75	30	2.8	1.901	2.060	0.7
27	-100	7	100	0	35	2.8	1.942	16.187	1.3
28	-100	7	75	25	35	2.8	1.945	15.500	1.1
29	-100	7	50	50	35	2.8	1.978	8.044	0.8
30	-100	7	25	75	35	2.8	2.019	1.864	0.5
31	-100	7	100	0	25	3.6	2.090	16.677	1.3
32	-100	7	75	25	25	3.6	2.075	16.481	0.9
33	-100	7	50	50	25	3.6	2.048	9.320	0.5
34	-100	7	25	75	25	3.6	1.958	1.864	1.0
35	-100	7	100	0	30	3.6	2.029	16.579	0.7
36	-100	7	75	25	30	3.6	2.008	15.696	0.6
37	-100	7	50	50	30	3.6	1.996	7.161	0.7
38	-100	7	25	75	30	3.6	2.030	1.766	0.4
39	-100	7	100	0	35	3.6	1.960	16.088	0.9
40	-100	7	75	25	35	3.6	1.980	14.715	0.8
41	-100	7	50	50	35	3.6	1.995	5.690	0.5
42	-100	7	25	75	35	3.6	2.010	1.472	0.4
43	-200	7	50	50	25	2.8	1.956	6.867	0.4
44	-200	7	25	75	25	2.8	1.961	2.551	0.4
45	-200	7	50	50	30	2.8	1.881	6.377	0.5

T 21

Serial No.	Mesh Size	Curing Days	Anhydrite (Parts)	Tailings (Parts)	Water (Parts)	FeSO4 (Parts)	Density (gr. cu.cm.)	U.C.S. (MPa)	Strain (%)
46	-200	7	25	75	30	2.8	1.996	1.766	0.3
47	-200	7	50	50	35	2.8	1.899	4.709	0.2
48	-200	7	25	75	35	2.8	2.025	2.354	0.6
49	-200	7	50	50	25	3.6	1.963	17.168	1.3
50	-200	7	25	75	25	3.6	1.963	4.611	1.2
51	-200	7	50	50	30	3.6	1.893	8.525	0.5
52	-200	7	25	75	30	3.6	1.973	2.060	0.2
53	-200	7	50	50	35	3.6	1.938	13.538	0.9
54	-200	7	25	75	35	3.6	1.973	1.864	0.3
55	-20	14	100	0	32	2.8	1.725	1.364	1.0
56	-20	14	75	25	32	2.8	1.853	4.120	0.6
57	-20	14	50	50	32	2.8	1.863	3.335	0.3
58	-20	14	25	75	32	2.8	1.799	0.392	0.2
59	-20	14	100	0	37	2.8	1.615	3.728	0.9
60	-20	14	75	25	37	2.8	1.810	3.630	0.4
61	-20	14	50	50	37	2.8	1.786	0.589	0.3
62	-20	14	25	75	37	2.8	1.810	0.491	0.2
63	-20	14	75	25	42	2.8	1.652	1.177	0.4
64	-20	14	50	50	42	2.8	1.708	1.373	0.6
65	-20	14	100	0	32	3.6	1.777	1.275	0.5
66	-20	14	75	25	32	3.6	1.910	7.358	0.6
67	-20	14	50	50	32	3.6	1.842	3.924	0.4
68	-20	14	25	75	32	3.6	1.727	0.196	0.2
69	-20	14	100	0	37	3.6	1.686	2.649	0.2
70	-20	14	75	25	37	3.6	1.736	1.177	0.3
71	-20	14	50	50	37	3.6	1.766	2.649	0.4
72	-20	14	100	0	42	3.6	1.507	1.079	0.6
73	-20	14	75	25	42	3.6	1.705	0.981	0.5
74	-20	14	50	50	42	3.6	1.783	1.864	0.5
75	-50	14	50	50	32	2.8	1.826	6.377	0.7
76	-50	14	25	75	32	2.8	1.759	1.079	0.5
77	-50	14	50	50	37	2.8	1.675	2.354	0.5
78	-50	14	25	75	37	2.8	1.732	0.589	0.4
79	-50	14	50	50	42	2.8	1.740	2.747	0.4
80	-50	14	50	50	32	3.6	1.827	7.712	0.7
81	-50	14	25	75	32	3.6	1.812	2.256	0.5
82	-50	14	50	50	37	3.6	1.798	5.886	0.5
83	-50	14	50	50	42	3.6	1.613	3.728	0.8
84	-50	14	25	75	42	3.6	1.724	1.079	0.6
85	-20	28	100	0	32	2.8	1.732	6.867	0.7
86	-20	28	75	25	32	2.8	1.882	16.677	0.5
87	-20	28	50	50	32	2.8	1.794	7.259	0.8
88	-20	28	25	75	32	2.8	1.748	1.766	0.7
89	-20	28	100	0	37	2.8	1.665	5.788	1.0
90	-20	28	75	25	37	2.8	1.739	8.437	0.8

T21

Serial No.	Mesh Size	Curing Days	Anhydrite (Parts)	Tailings (Parts)	Water (Parts)	FeSO4 (Parts)	Density (gr. cu.cm.)	U.C.S. (MPa)	Strain (%)
91	-20	28	50	50	37	2.8	1.712	3.199	0.6
92	-20	28	100	0	42	2.8	1.668	2.849	0.7
93	-20	28	75	25	42	2.8	1.812	5.592	0.7
94	-20	28	50	50	42	2.8	1.725	5.396	1.0
95	-20	28	25	75	42	2.8	1.702	0.883	0.5
96	-20	28	100	0	32	3.6	1.701	3.826	0.7
97	-20	28	75	25	32	3.6	1.793	7.554	1.0
98	-20	28	50	50	32	3.6	1.774	5.984	0.7
99	-20	28	25	75	32	3.6	1.721	1.962	0.3
100	-20	28	100	0	37	3.6	1.677	3.728	0.8
101	-20	28	75	25	37	3.6	1.755	6.377	0.9
102	-20	28	50	50	37	3.6	1.745	3.630	0.9
103	-20	28	25	75	37	3.6	1.714	0.687	0.3
104	-20	28	100	0	42	3.6	1.562	2.158	0.6
105	-20	28	75	25	42	3.6	1.682	5.396	1.0
106	-20	28	50	50	42	3.6	1.729	5.592	1.3
107	-50	28	100	0	32	2.8	1.688	15.696	1.0
108	-50	28	75	25	32	2.8	1.657	15.206	0.9
109	-50	28	50	50	32	2.8	1.635	9.516	0.9
110	-50	28	25	75	32	2.8	1.693	3.041	0.9
111	-50	28	100	0	37	2.8	1.678	11.282	1.0
112	-50	28	75	25	37	2.8	1.624	10.889	0.9
113	-50	28	50	50	37	2.8	1.610	8.927	0.8
114	-50	28	25	75	37	2.8	1.663	2.256	0.6
115	-50	28	100	0	42	2.8	1.599	11.282	0.5
116	-50	28	75	25	42	2.8	1.589	9.123	0.8
117	-50	28	50	50	42	2.8	1.567	5.101	0.8
118	-50	28	25	75	42	2.8	1.591	1.668	0.8
119	-50	28	100	0	32	3.6	1.700	15.990	1.2
120	-50	28	75	25	32	3.6	1.668	12.655	1.1
121	-50	28	50	50	32	3.6	1.667	6.377	0.8
122	-50	28	25	75	32	3.6	1.680	3.237	0.7
123	-50	28	100	0	37	3.6	1.624	9.418	0.9
124	-50	28	75	25	37	3.6	1.578	11.183	0.6
125	-50	28	50	50	37	3.6	1.705	8.731	0.6
126	-50	28	25	75	37	3.6	1.684	3.434	0.5
127	-50	28	100	0	42	3.6	1.689	14.319	0.6
128	-50	28	75	25	42	3.6	1.540	9.516	0.9
129	-50	28	50	50	42	3.6	1.594	7.259	0.6
130	-50	28	25	75	42	3.6	1.603	1.864	0.5
131	-100	28	100	0	25	2.8	2.003	15.892	1.0
132	-100	28	75	25	25	2.8	1.932	16.579	0.7
133	-100	28	50	50	25	2.8	1.925	15.892	0.7
134	-100	28	25	75	25	2.8	1.849	5.690	0.4
135	-100	28	100	0	30	2.8	1.852	16.579	1.0

T22

Serial No.	Mesh Size	Curing Days	Anhydrite (Parts)	Tailings (Parts)	Water (Parts)	FeSO4 (Parts)	Density (gr./cu.cm.)	U.C.S. (MPa)	Strain (%)
136	-100	28	75	25	30	2.8	1.863	16.481	0.8
137	-100	28	50	50	30	2.8	1.870	15.794	0.5
138	-100	28	25	75	30	2.8	1.823	3.728	0.5
139	-100	28	100	0	35	2.8	1.817	17.168	0.6
140	-100	28	75	25	35	2.8	1.796	17.168	0.6
141	-100	28	50	50	35	2.8	1.862	16.187	0.8
142	-100	28	25	75	35	2.8	1.755	3.630	0.3
143	-100	28	100	0	25	3.6	1.927	16.971	2.6
144	-100	28	75	25	25	3.6	1.941	16.677	1.5
145	-100	28	50	50	25	3.6	1.894	16.481	0.7
146	-100	28	25	75	25	3.6	1.908	7.358	0.4
147	-100	28	100	0	30	3.6	1.930	16.677	1.0
148	-100	28	75	25	30	3.6	1.856	17.364	0.8
149	-100	28	50	50	30	3.6	1.808	15.892	0.7
150	-100	28	25	75	30	3.6	1.789	3.332	0.7
151	-100	28	100	0	35	3.6	1.816	16.775	1.5
152	-100	28	75	25	35	3.6	1.828	14.911	0.9
153	-100	28	50	50	35	3.6	1.831	13.440	0.9
154	-100	28	25	75	35	3.6	1.832	1.766	0.4
155	-200	28	100	0	25	2.8	1.856	12.749	2.1
156	-200	28	75	25	25	2.8	1.916	13.342	1.8
157	-200	28	50	50	25	2.8	1.872	11.576	1.0
158	-200	28	25	75	25	2.8	1.888	6.671	0.7
159	-200	28	100	0	30	2.8	1.841	16.481	0.8
160	-200	28	75	25	30	2.8	1.724	15.990	0.8
161	-200	28	50	50	30	2.8	1.744	15.304	0.7
162	-200	28	25	75	30	2.8	1.837	3.630	0.7
163	-200	28	100	0	35	2.8	1.703	18.443	0.4
164	-200	28	75	25	35	2.8	1.712	17.462	0.6
165	-200	28	50	50	35	2.8	1.807	18.345	0.6
166	-200	28	25	75	35	2.8	1.732	9.320	0.5
167	-200	28	100	0	25	3.6	1.946	17.756	0.7
168	-200	28	75	25	25	3.6	1.851	17.560	0.8
169	-200	28	50	50	25	3.6	1.841	17.560	0.7
170	-200	28	25	75	25	3.6	1.811	9.516	0.4
171	-200	28	100	0	30	3.6	1.778	15.990	0.7
172	-200	28	75	25	30	3.6	1.721	15.794	0.4
173	-200	28	50	50	30	3.6	1.794	16.383	0.7
174	-200	28	25	75	30	3.6	1.803	7.259	0.4
175	-200	28	100	0	35	3.6	1.722	16.971	1.1
176	-200	28	75	25	35	3.6	1.694	16.775	0.8
177	-200	28	50	50	35	3.6	1.795	16.677	0.5
178	-200	28	25	75	35	3.6	1.762	5.396	0.3

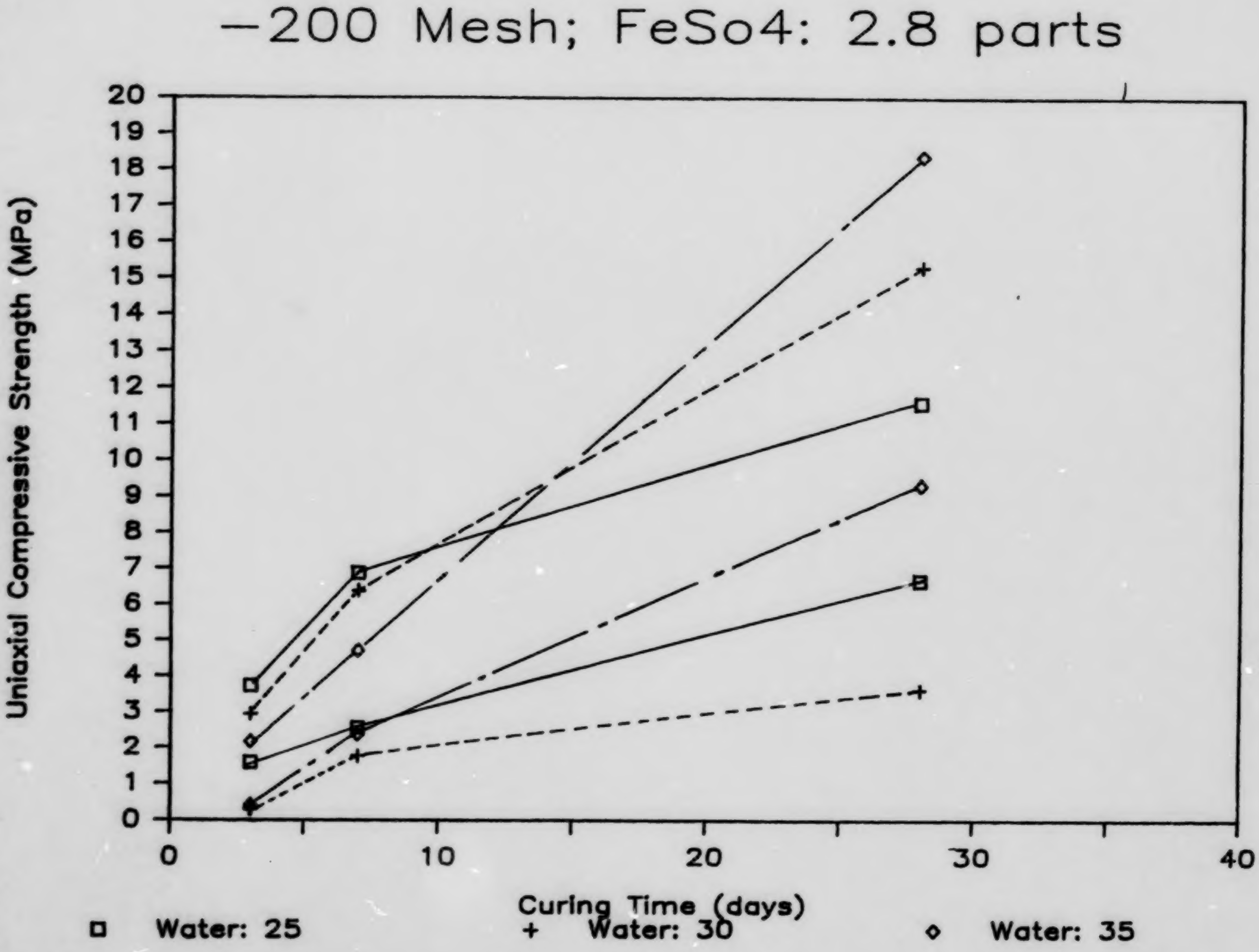


Fig. 52 - Comparison of various ANTAIL mixes using 2.8 parts by weight accelerate and -200 mesh anhydrite.

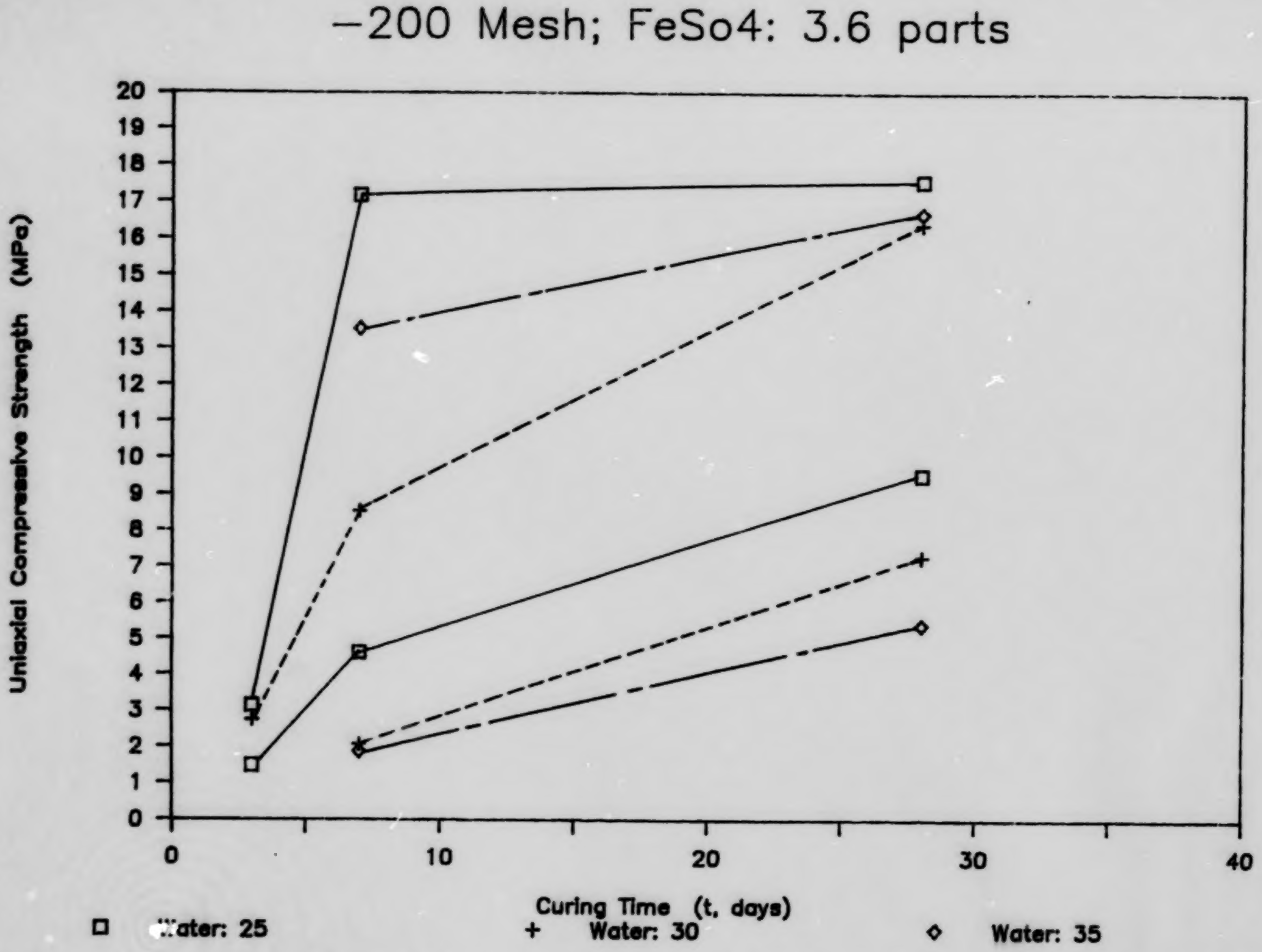


Fig. 53 - Comparison of various ANTAIL mixes using 3.6 parts by weight accelerator and -200 mesh anhydrite.

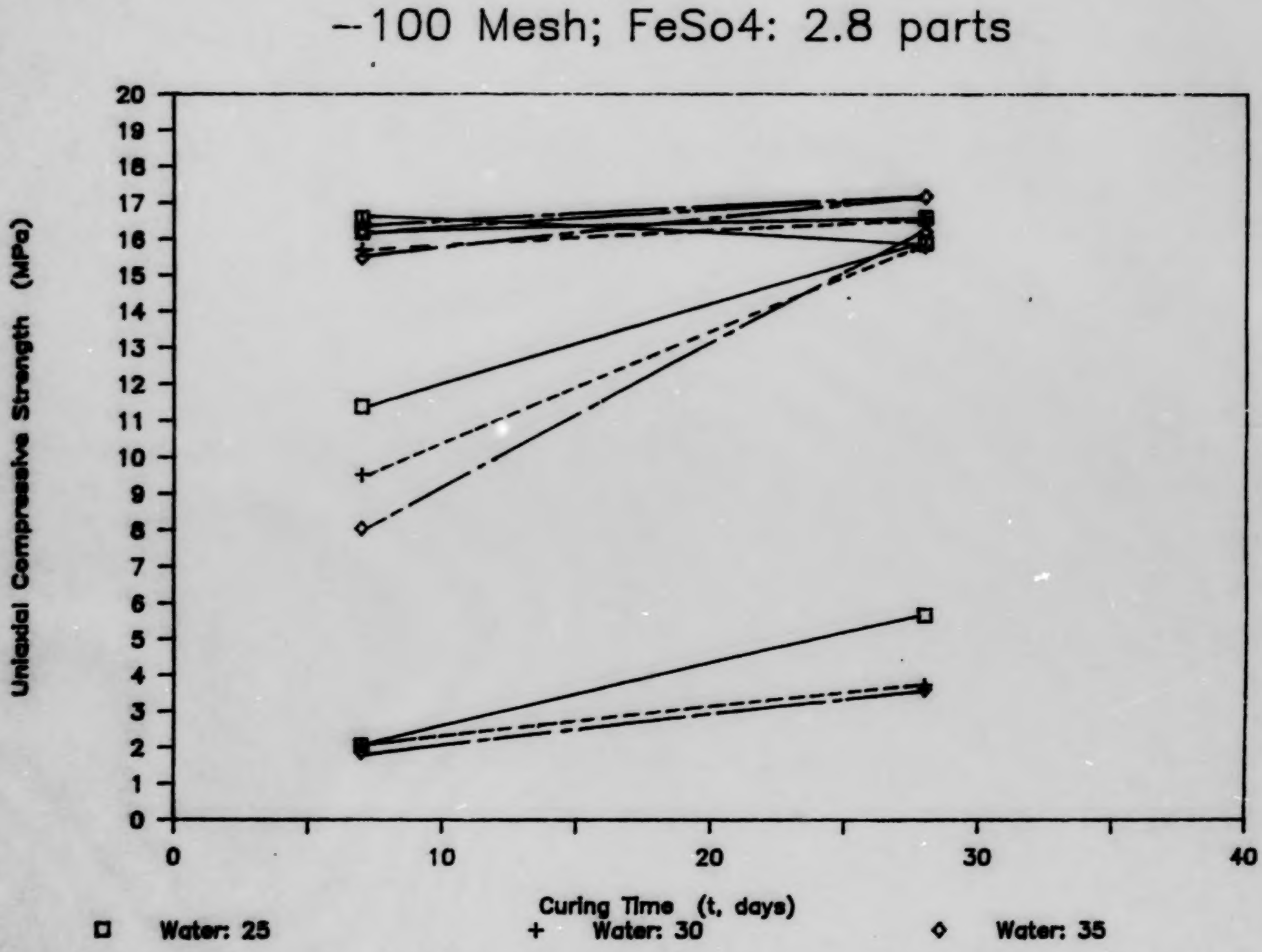


Fig. 54 - Comparison of various ANTAIL mixes using 2.8 parts by weight accelerator and -100 mesh anydrite.

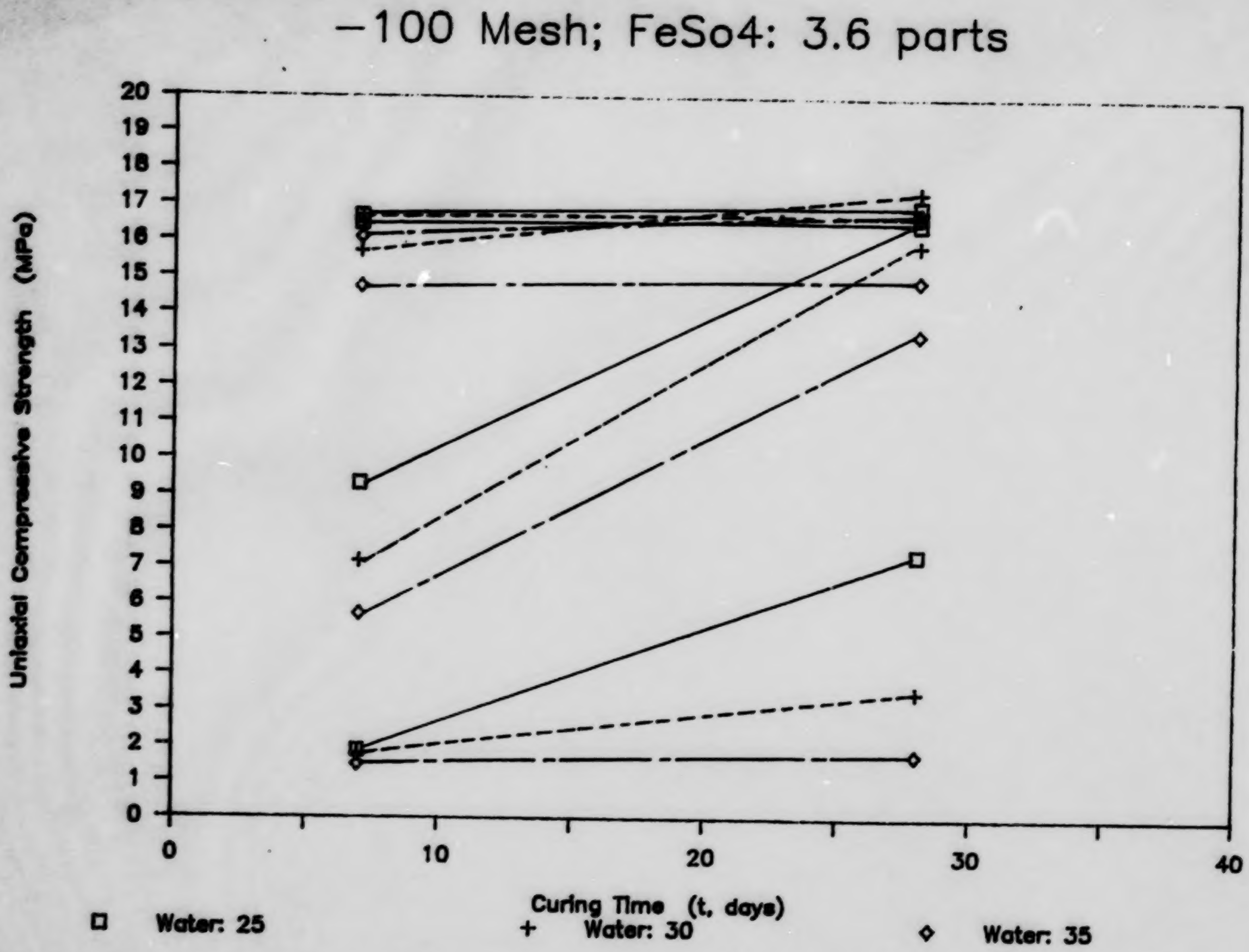


Fig. 55 - Comparison of various ANTAIL mixes using 3.6 parts by weight accelerator and -100 mesh anhydrite.

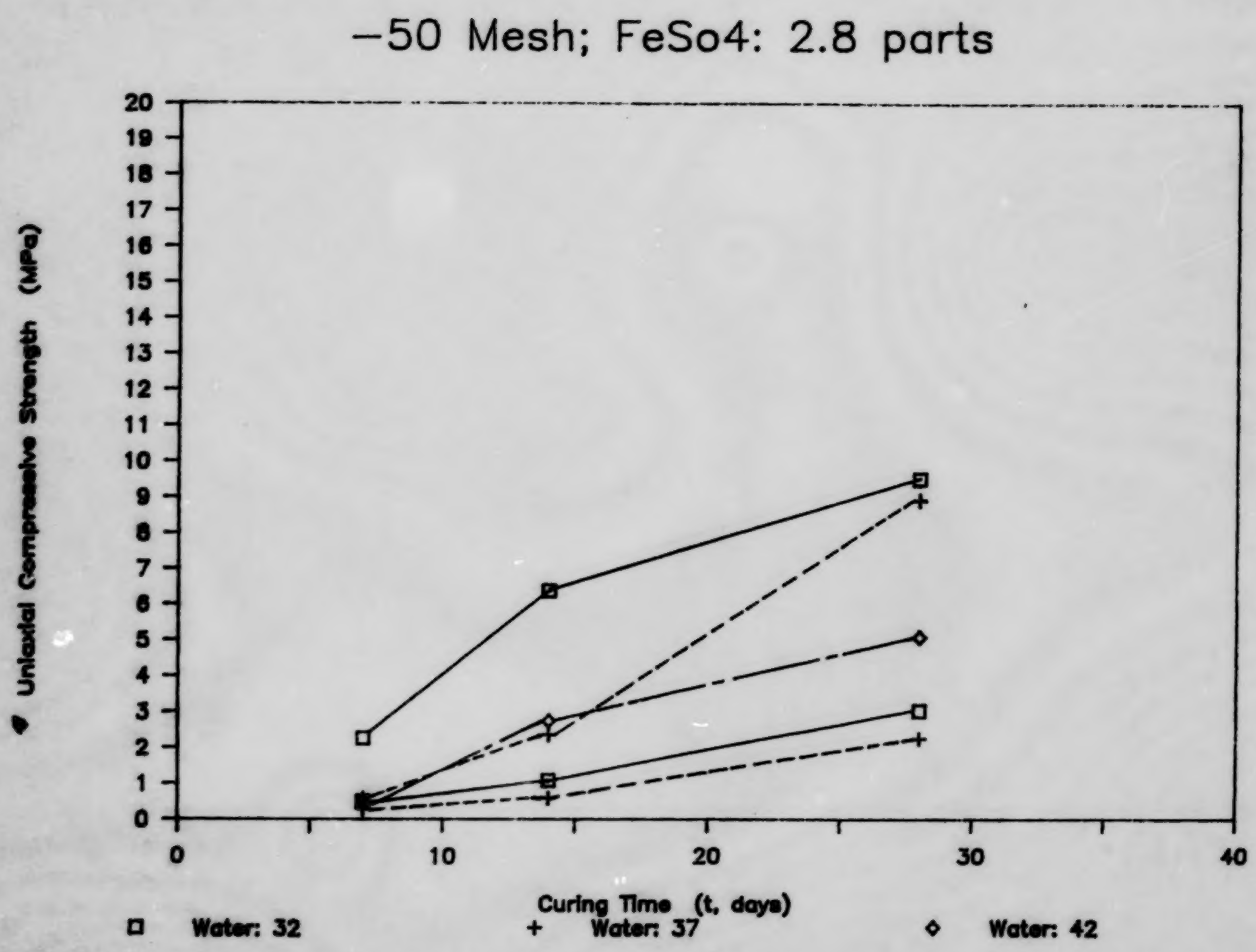


Fig. 56 - Comparison of various ANTAIL mixes using 2.8 parts by weight accelerator and -50 to +100 mesh anhydrite.

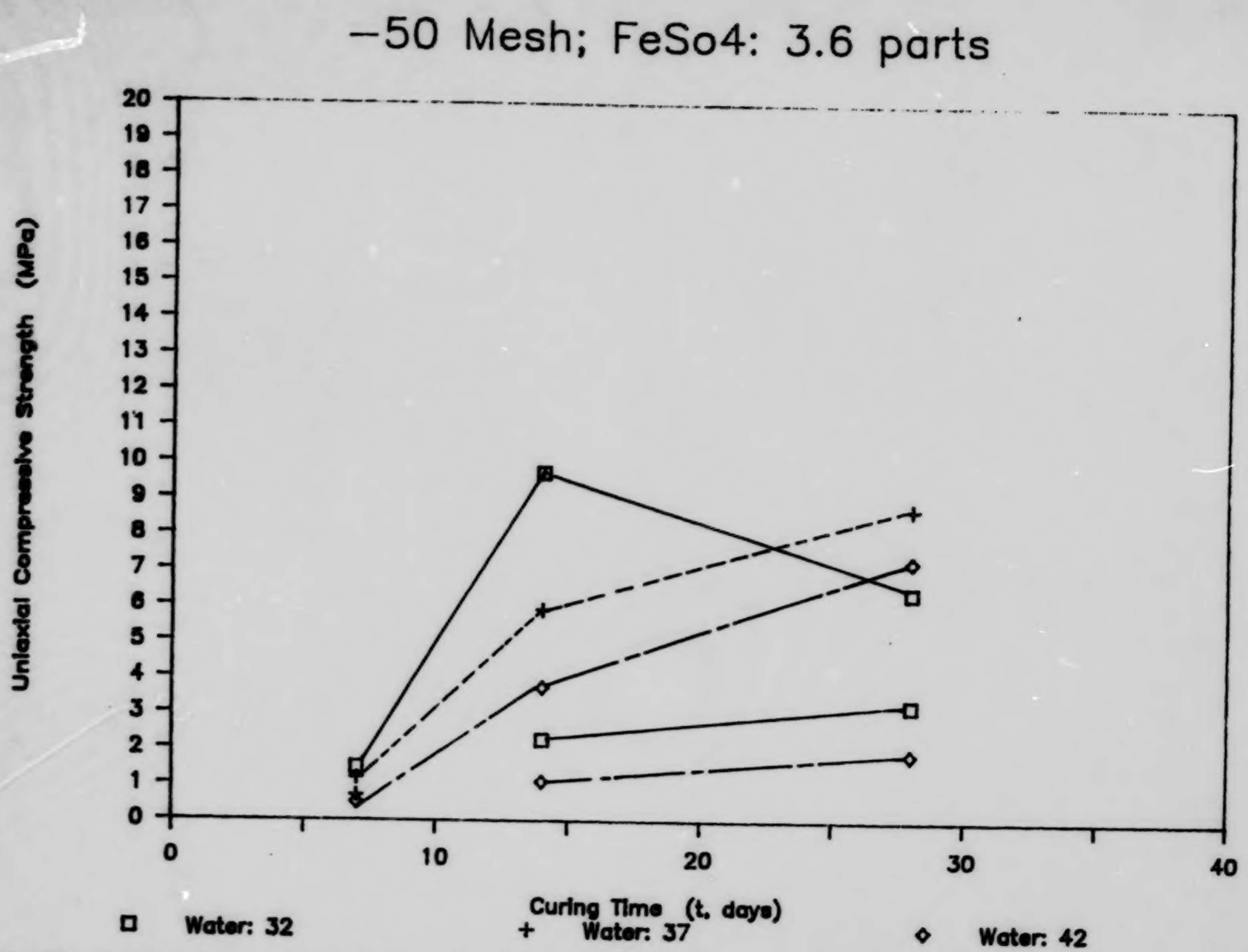


Fig. 57 - Comparison of various ANTAIL mixes using 3.6 parts by weight accelerator and -50 to +100 mesh anhydrite.

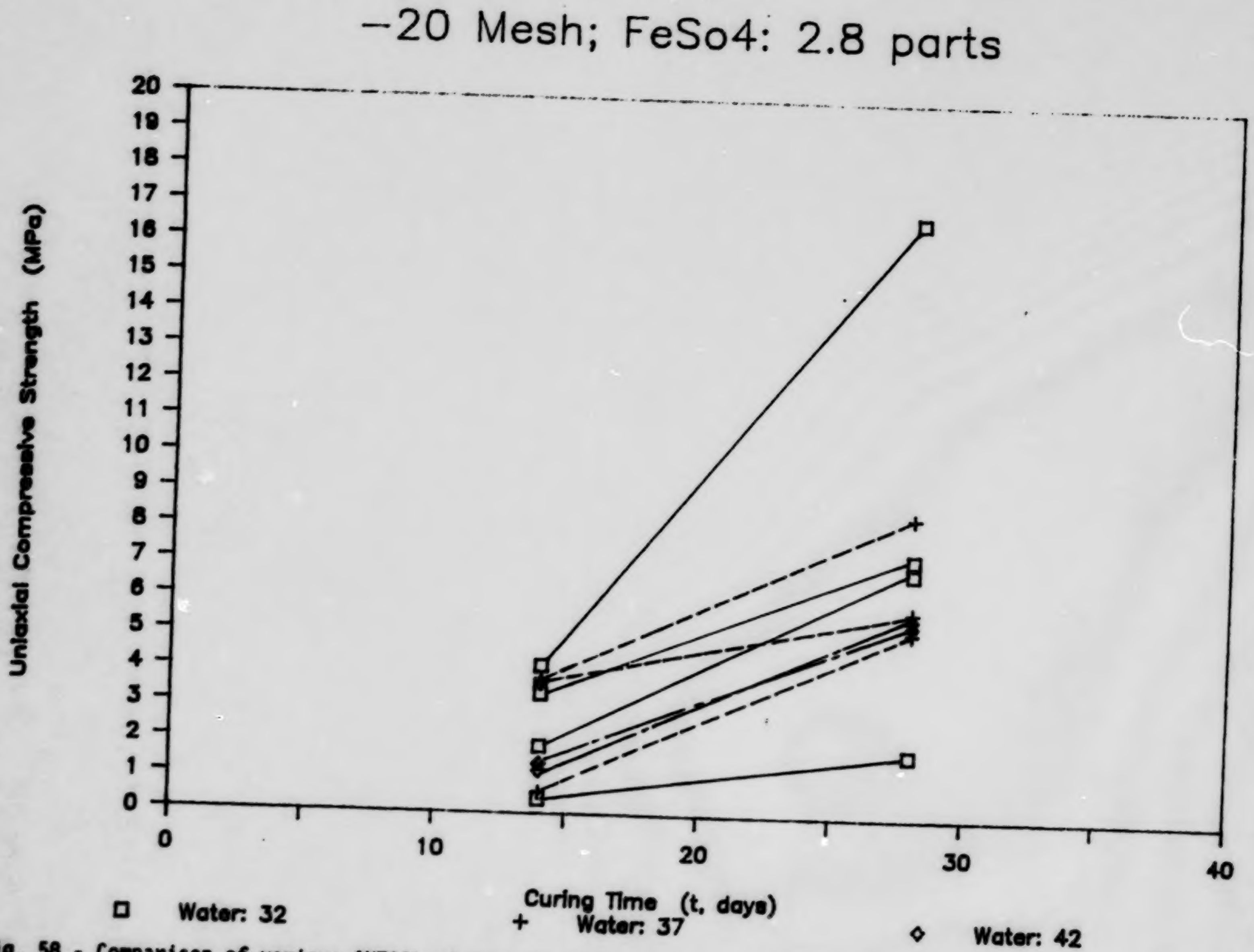


Fig. 58 - Comparison of various ANTAIL mixes using 2.8 parts by weight accelerator and -20 to +50 mesh anhydrite.

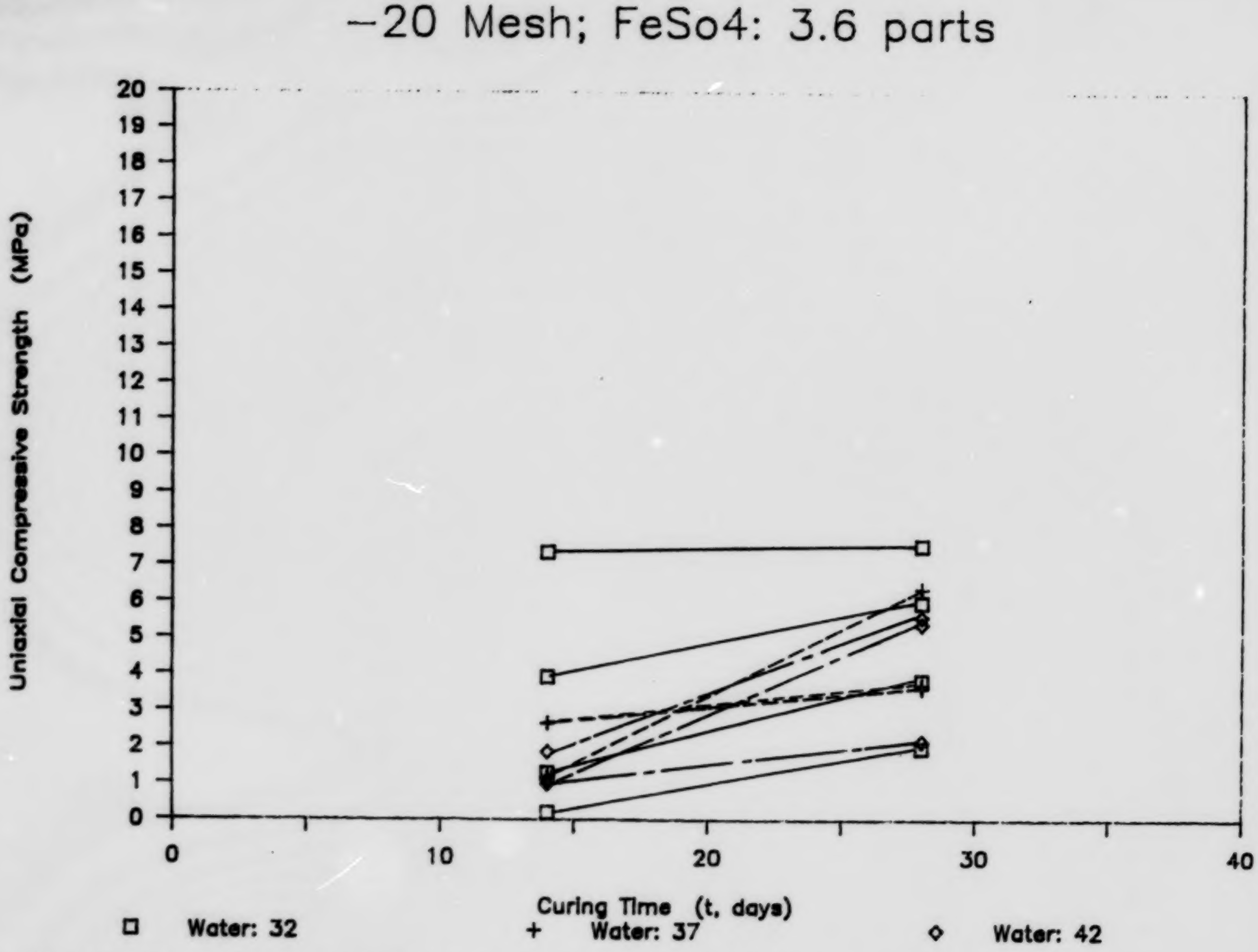


Fig. 59 - Comparison of various ANTAIL mixes using 3.6 parts by weight accelerator and -20 to +50 mesh anhydrite.

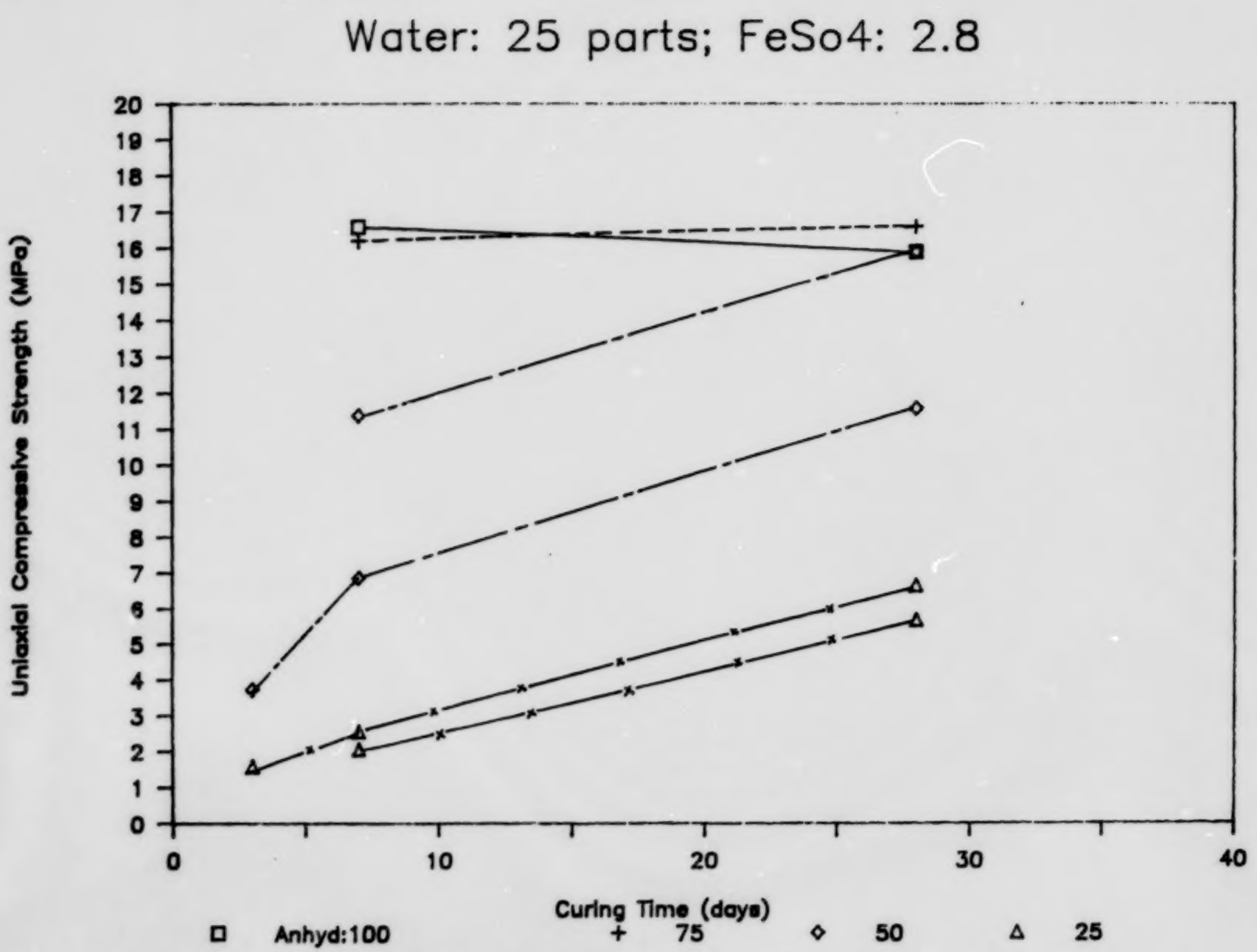


Fig. 60 - Comparison of various ANTAIL mixes using 2.8 accelerator and 25 parts by weight water.

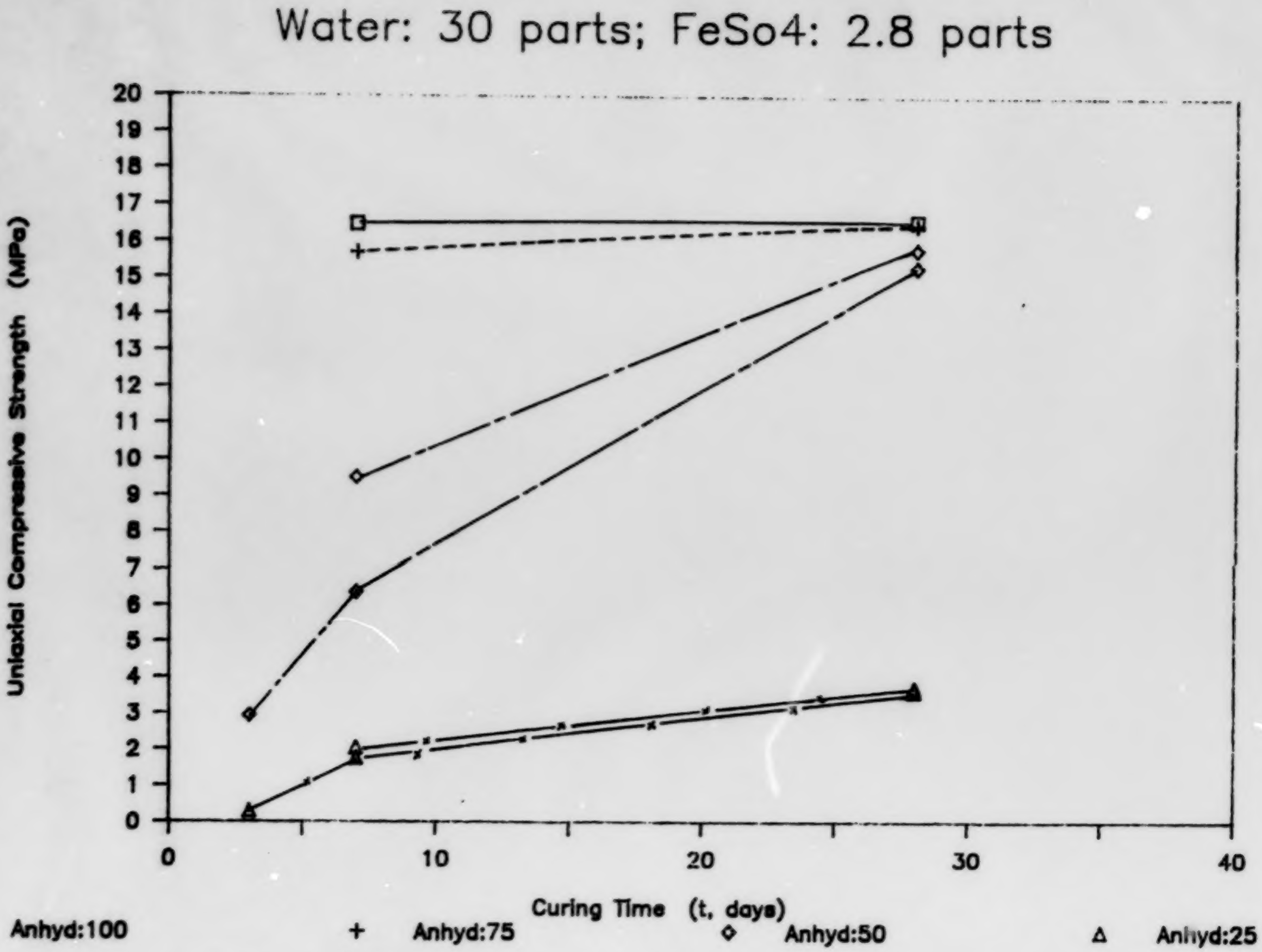
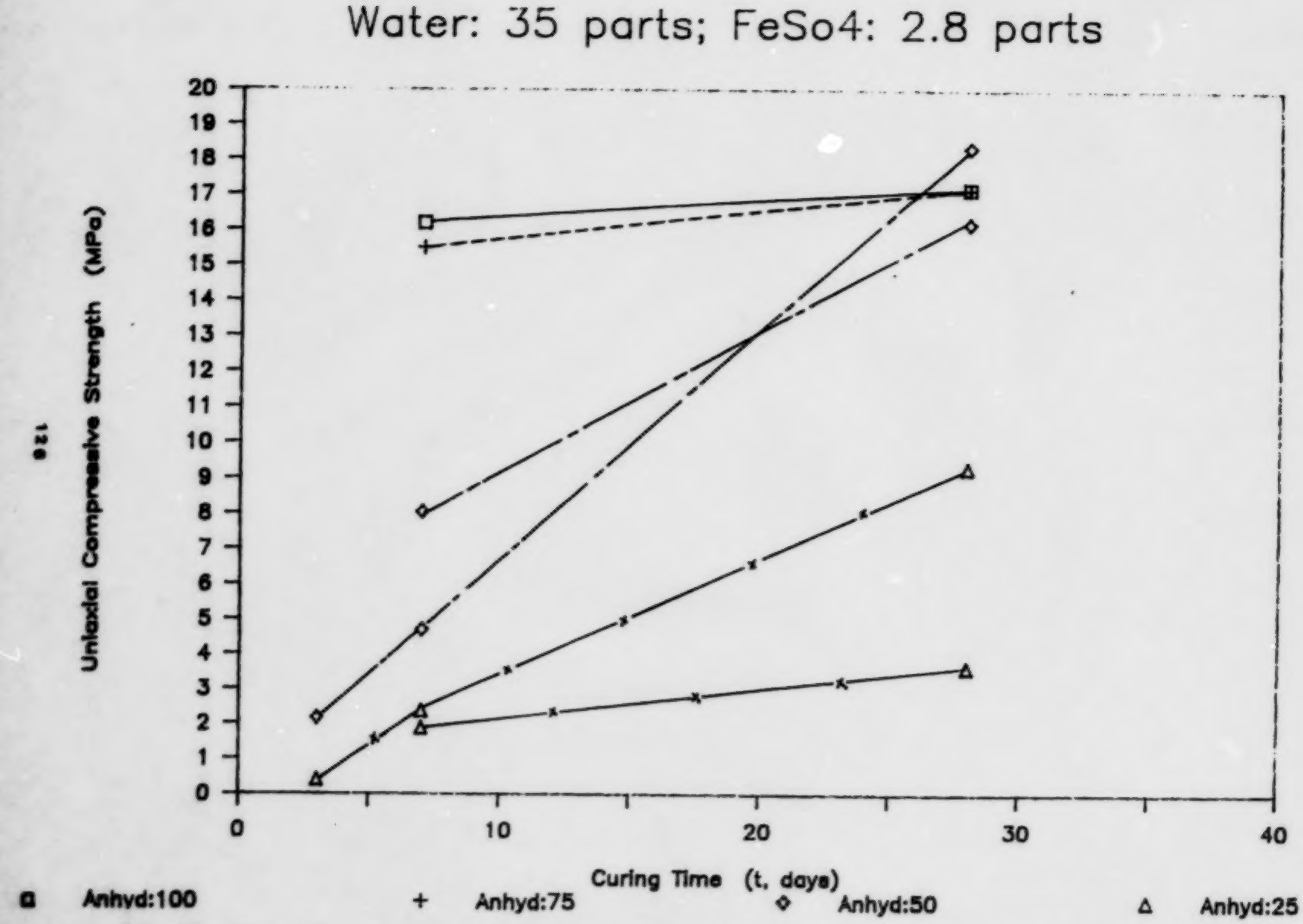


Fig. 61 - Comparison of various ANTAIL mixes using 2.8 acceleration and 30 parts by weight water.



Fig, 62 - Comparison of various ANTAIL mixes using 2.8 acelerator and 35 parts by weight water.

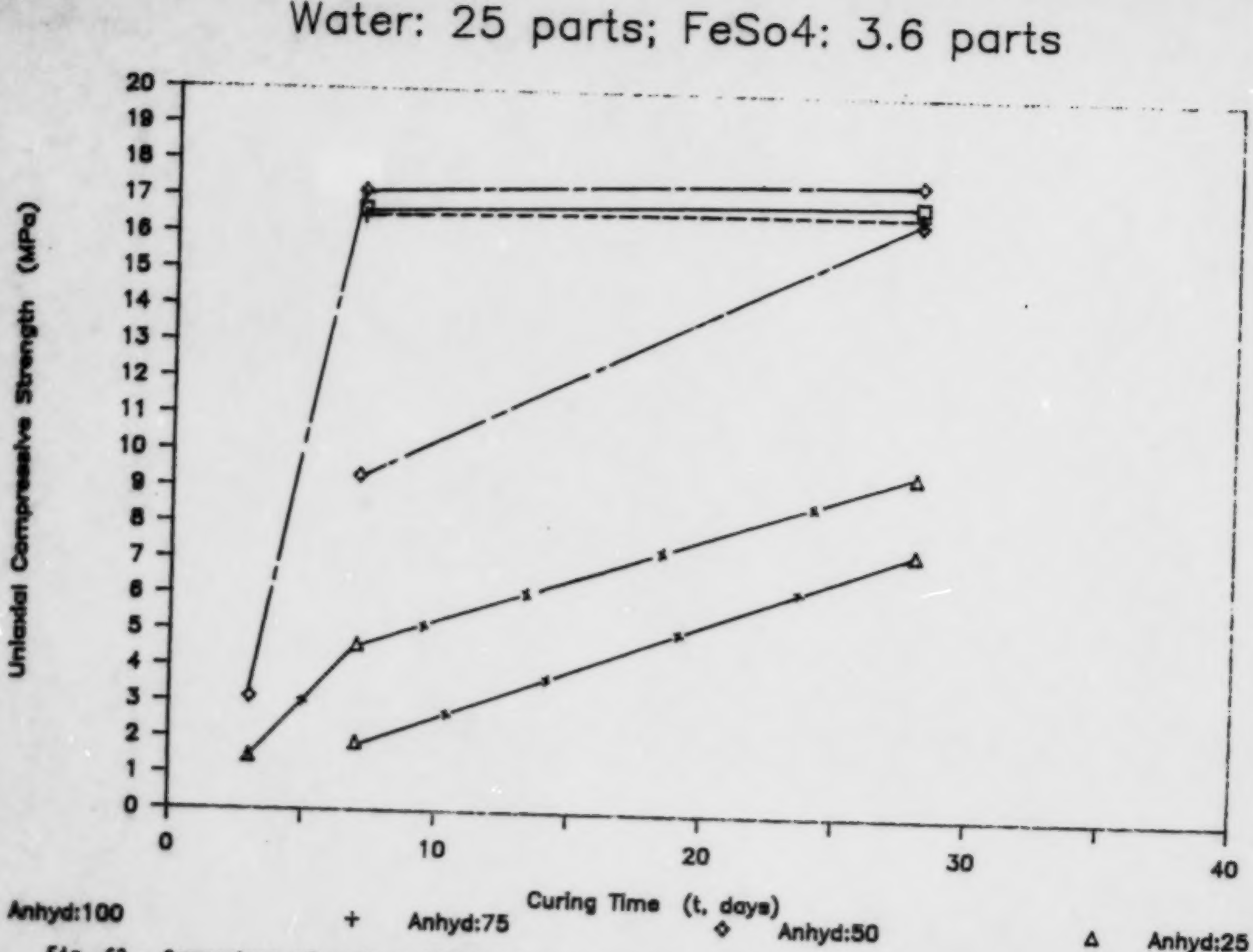

Water: 25 parts; FeSo4: 3.6 parts

Fig. 63 - Comparison of various ANTAIL mixes using 3.6 accelerator and 25 parts by weight water.

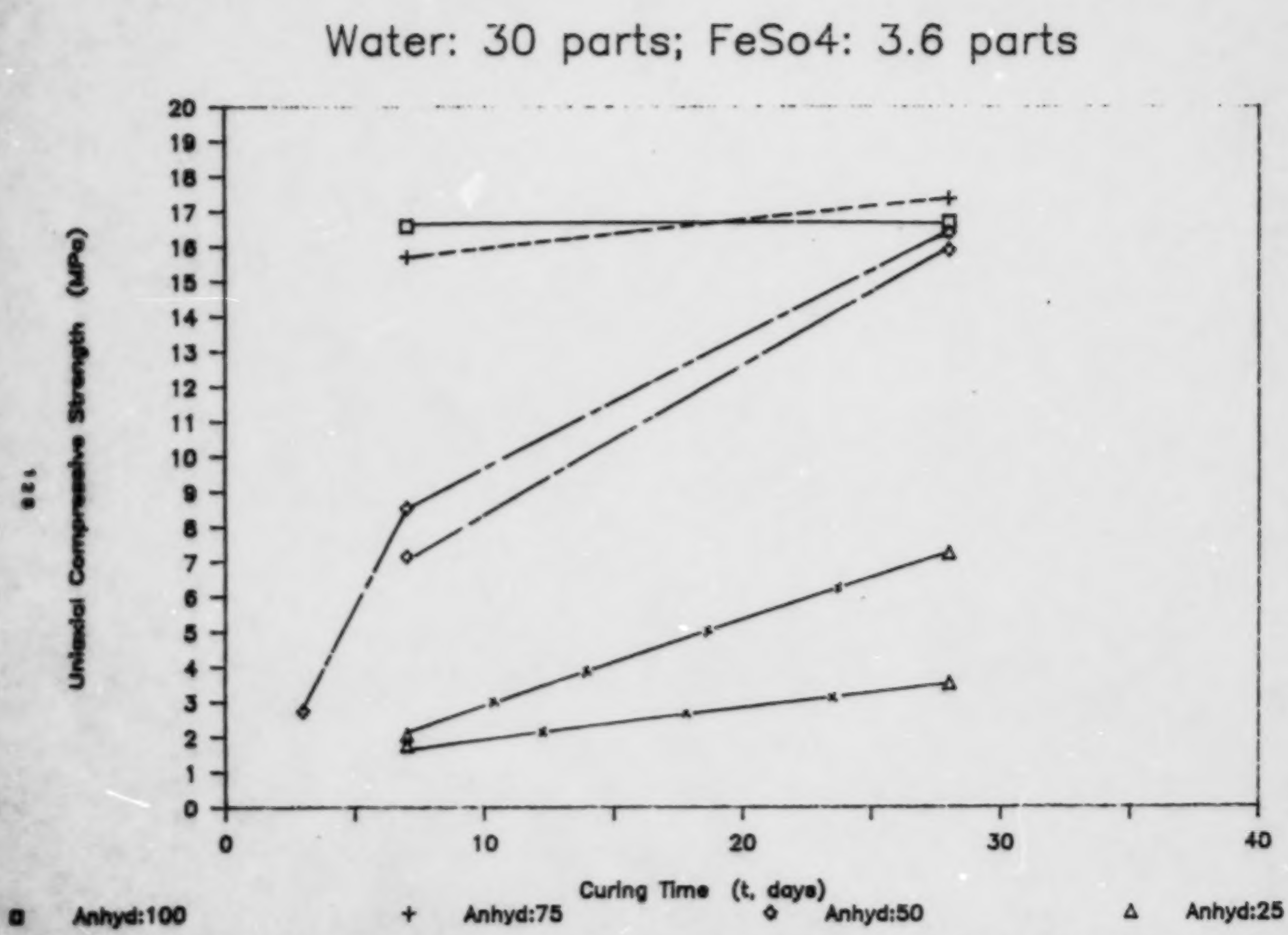


Fig. 64 - Comparison of various ANTAIL mixes using 3.6 accelerator and 30 parts by weight water.

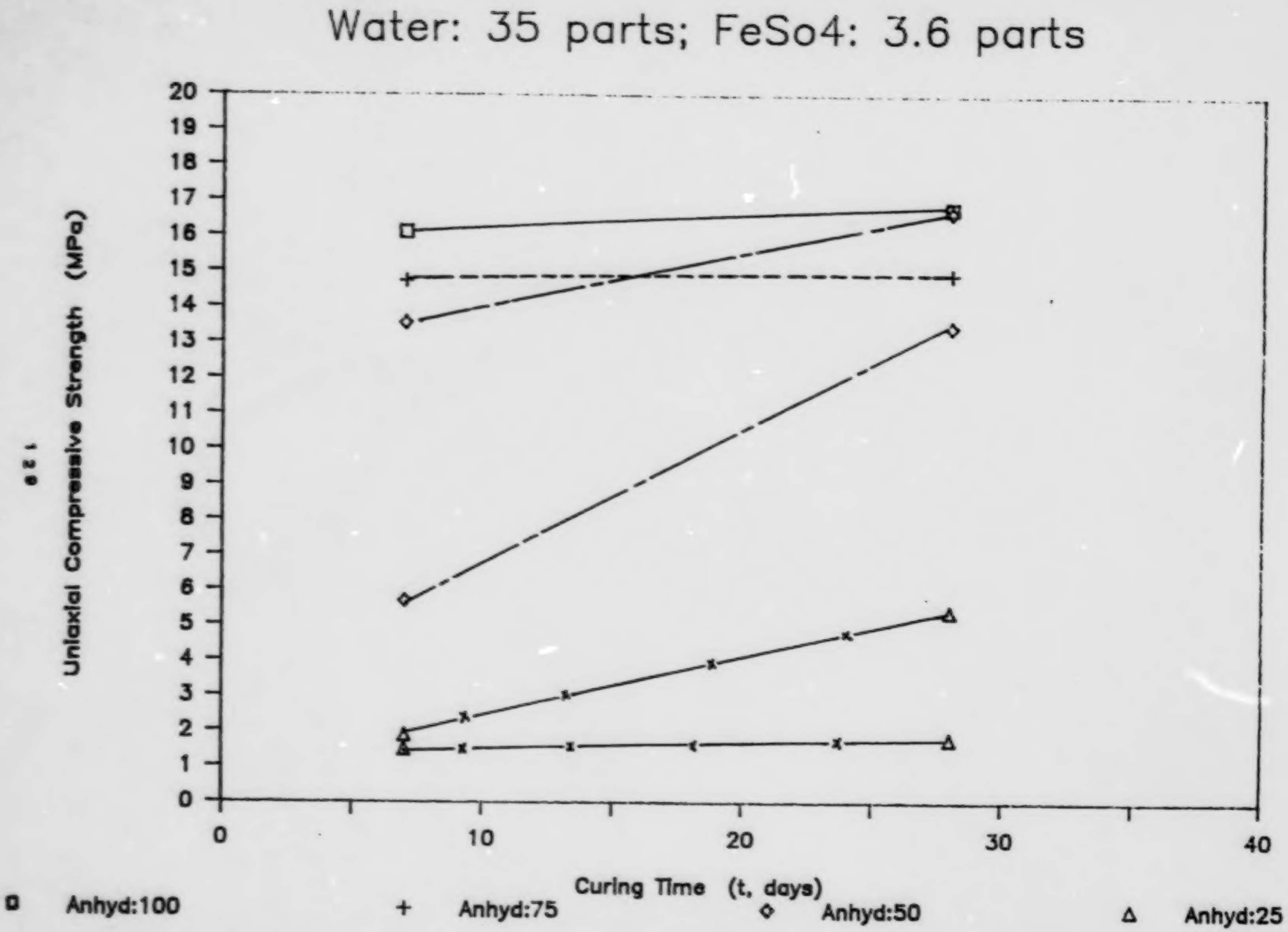


Fig. 65 - Comparison of various ANTAIL mixes using 3.6 accelerator and 35 parts by weight water.

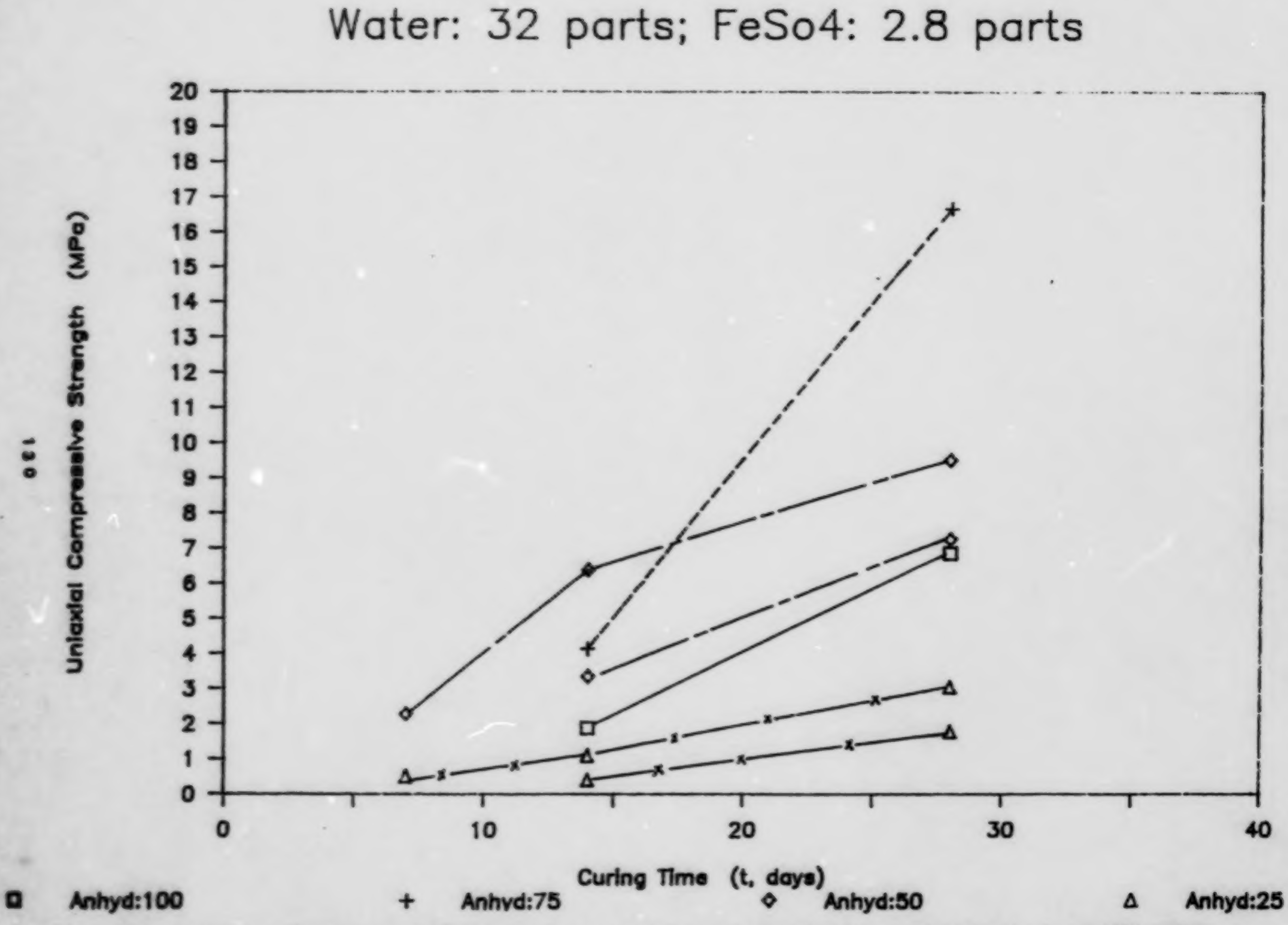


Fig. 66 - Comparison of various ANTAIL mixes using 2.8 parts accelerator and 32 parts by weight water.

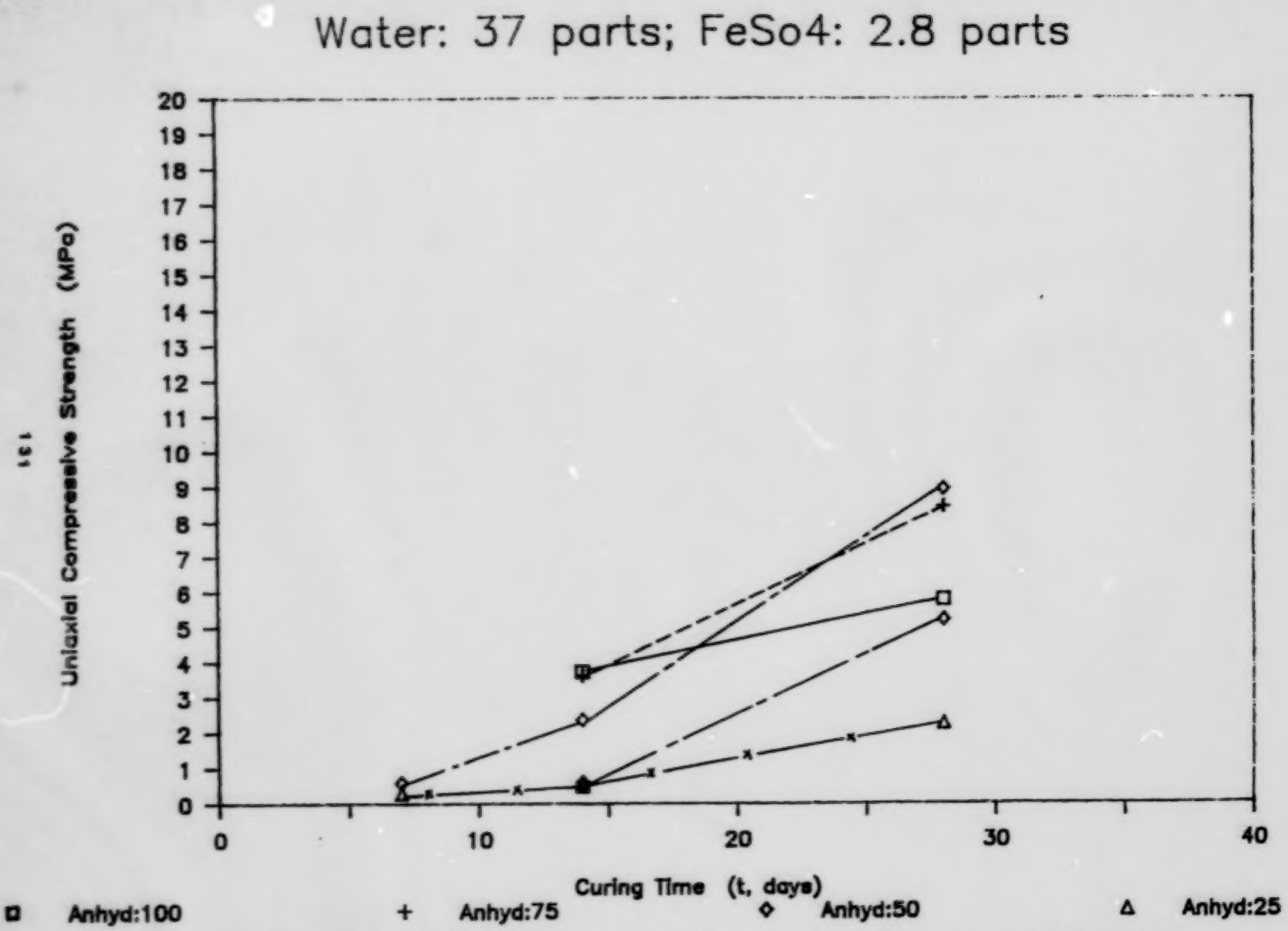


Fig. 67 - Comparison of various ANTAIL mixes using 2.8 accelerator and 37 parts by weight water.

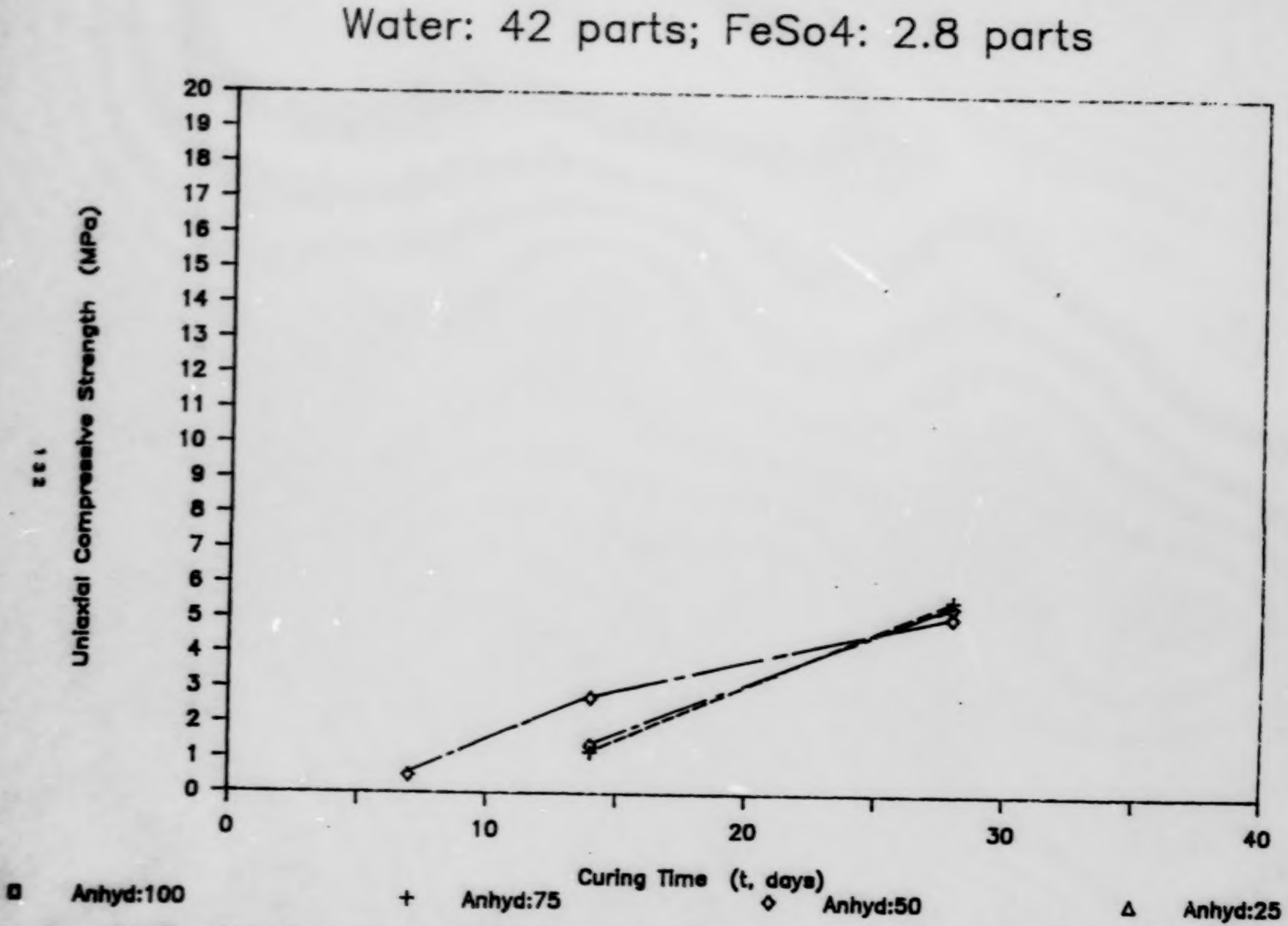


Fig. 68 - Comparison of various ANTAIL mixes using 2.8 accelerator and 42 parts by weight water.

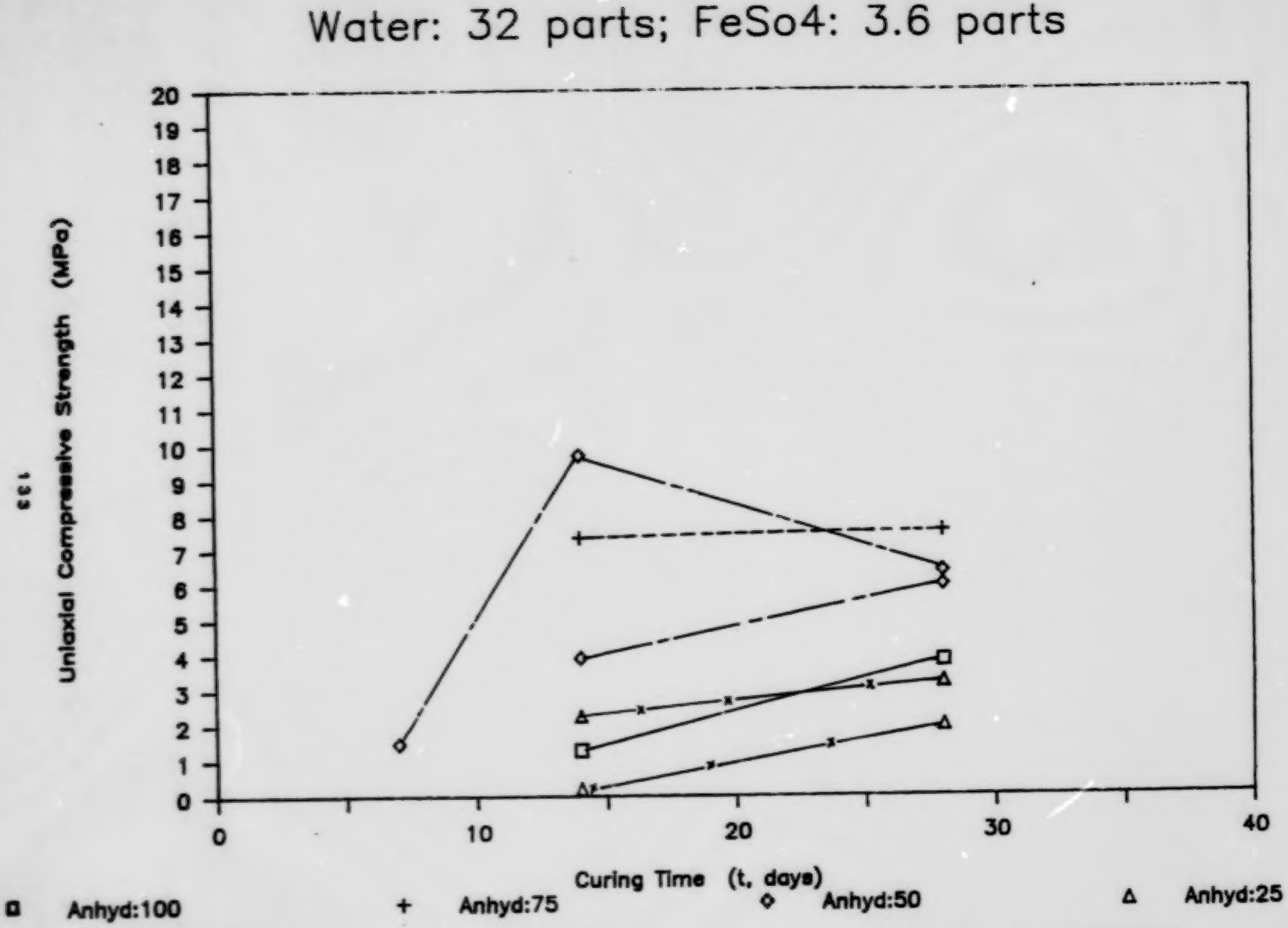


Fig. 69 - Comparison of various ANTAIL mixes using 3.6 accelerator and 32 parts by weight water.

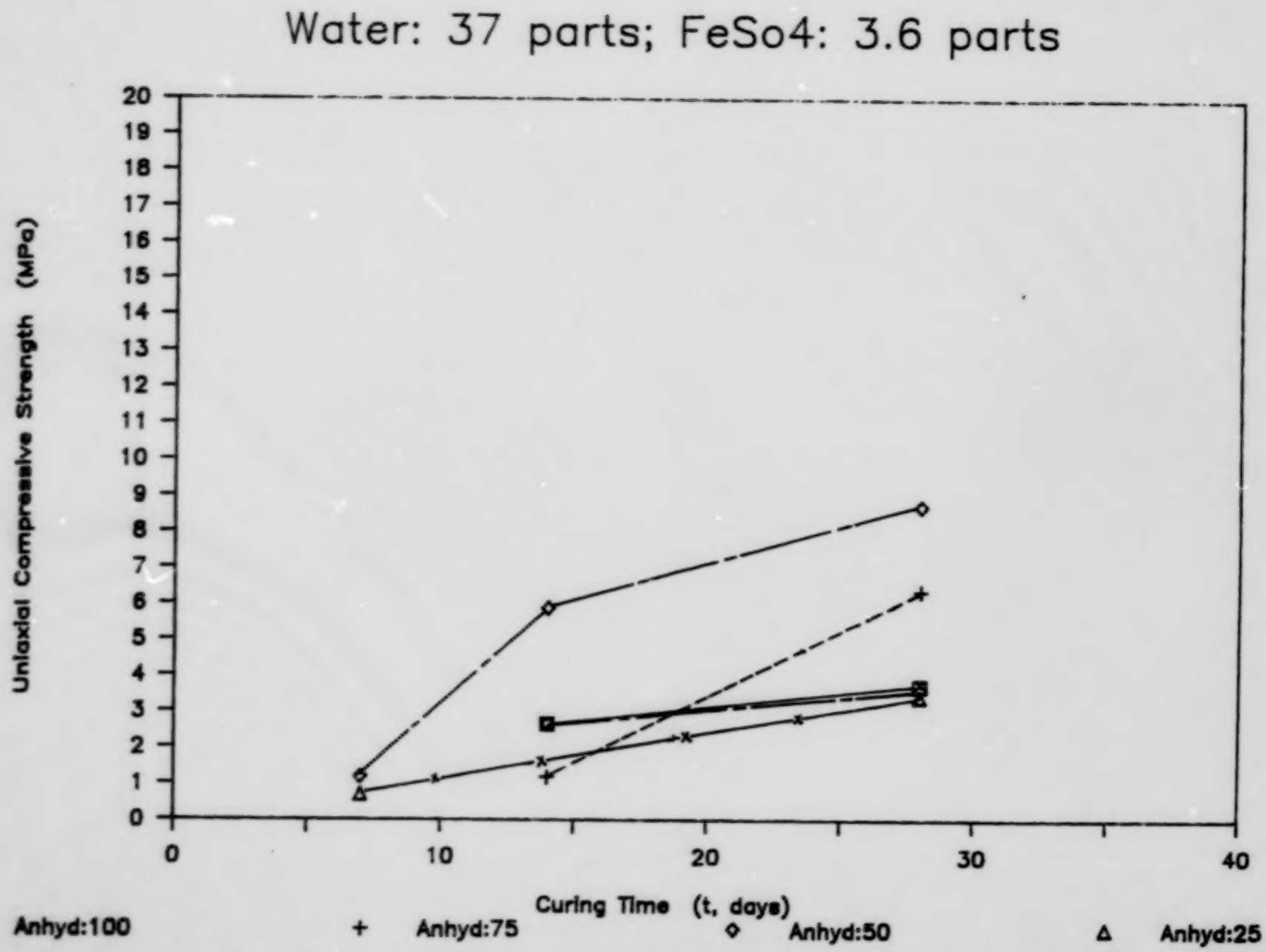


Fig. 70 - Comparison of various ANTAIL mixes using 3.6 accelerator and 37 parts by weight water.

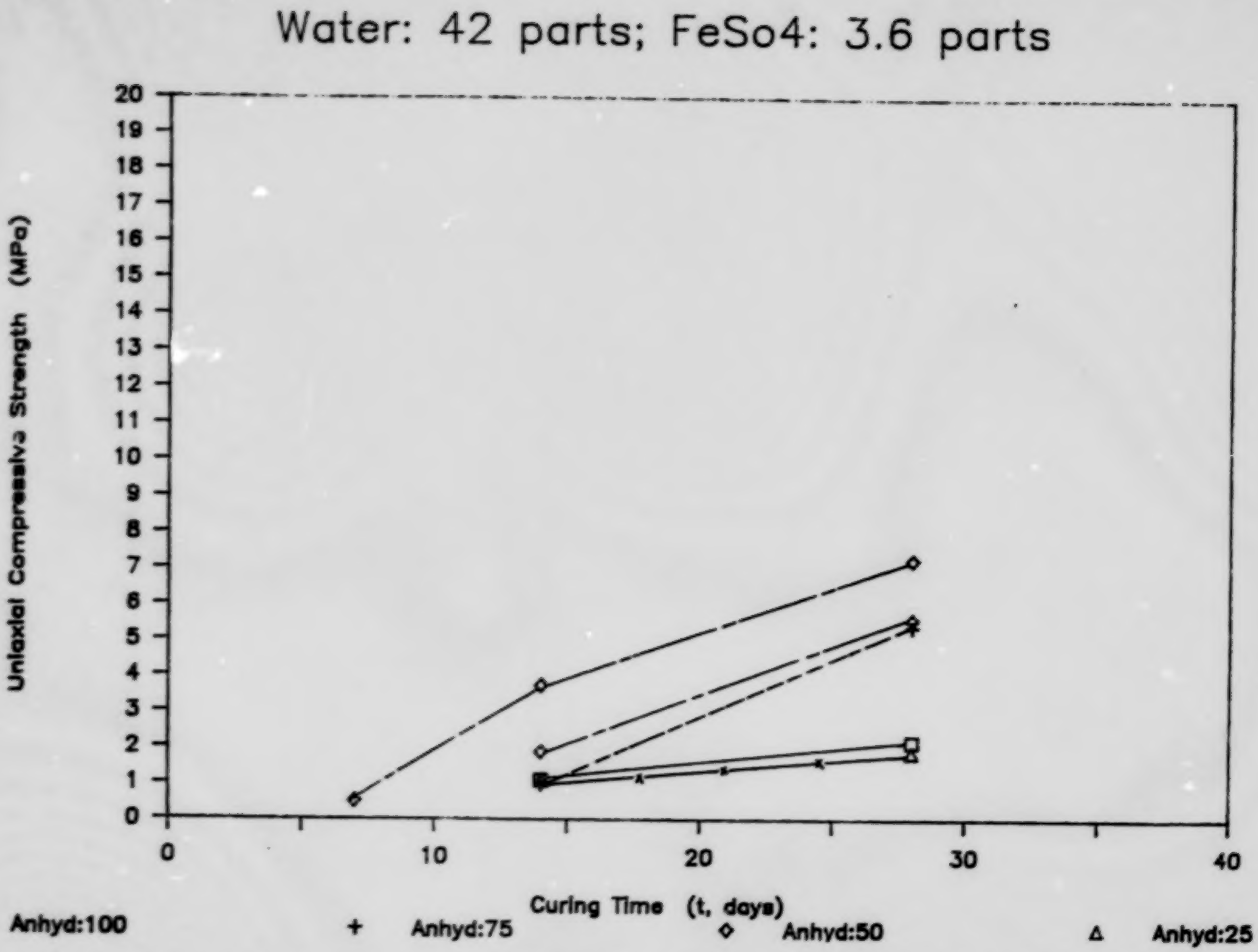


Fig. 71 - Comparison of various ANTAIL mixes using 3.6 accelerator and 42 parts by weight water.

(e) Pumpability of the mix can be ensured with 25 parts water by weight. This holds even for the largest sizes of the anhydrite tested, i.e. -20 to +50 mesh.

(f) Within a fluctuation of 25 to 30 parts by weight water in the mixtures, the U.C.S. does not change appreciably (Figs. 60 to 64). However, increasing the water content to 42 parts by weight (i.e. 41% increase), resulted 45% decrease in the U.C.S., on average (Figs. 65 to 71).

(g) The recommended mixture of ANTAIL within the U.C.S. of 15.9 MPa in 28 days is found to have an average density (γ) of 1900 kg/m^3 and the following parts by weight constituents:

-100 mesh Anhydrite : Tailing : $FeSO_4$, H_2O : Water
50 : 50 : 2.8 : 25

(h) The above noted mixture with anhydrite size of -50 to +100 mesh had a density (γ) of 1600 kg/m^3 and a U.C.S. of 8.9 MPa in 28 days.

(i) Using -200 mesh anhydrite is not recommended due to excessive cost of milling the rock. Although it increases the U.C.S. of the ANTAIL.

(j) The Stress/Strain results confirms the conclusions of (a) to (i), see Appendices A15-A102.

(k) The -100 mesh anhydrite mix, noted in the paragraph (g), displayed an average Modulus of Elasticity (E) of 1.9 GPa.

(l) The -50 to +100 mesh anhydrite mix, noted in the paragraph (h), displayed an average E-value of 1.1 GPa.

CONCLUSIONS

Amongst the various pumpable mixtures tested, the most suitable combination for the tight backfill (tightfill) in the order of performance and cost effectiveness are as follows:

1 - ANTAIL - by weight ratio of:

-50 to +100 mesh Nat. Anhydrite : Tailing : Accelerator : Water

50 : 50 : 2.8 : 25

Density = 1600 kg/m^3, after 28 days of curing
U.C.S. = 8.9 MPa
E = 1.1 GPa.

2 - ANTAIL - The second least expensive, by weight ratio of:

-100 mesh Nat. Anhydrite : Tailing : Accelerator : Water

25 : 75 : 2.8 : 25

Density = 1849 kb/m^3, after 28 days of curing
U.C.S. = 5.7 MPa
E = 1.4 GPa.

3 - ANTAIL - The most expensive and best performing of ANTAIL group, by weight ratio of:

-100 mesh Nat. Anhydrite : Tailing : Accelerator : Water

50 : 50 : 2.8 : 25

Density = 1900 kg/m^3, after 28 days of curing
U.C.S. = 15.9 MPa
E = 1.9 GPa.

4 - **ANPAK** - Has high transportation cost, by weight ratio of:

-100 mesh Nat. Anhydrite : Accelerator : Water

100 : 2.8 : 10

Density = 2292 kg/m^3, after 28 days of curing.
U.C.S. = 11 MPa
E = 1.1 GPa.

5 - **ANTAIL** - The least expensive and poor performing, by weight ratio of:

-50 to +100 mesh Nat. Anhydrite : Tailing : Accelerator : Water

25 : 75 : 2.8 : 32

Density = 1693 kg/m^3, after 28 days of curing
U.C.S. = 3.0 MPa
E = 0.34 GPa.

6 - **ANFEL** - The second most expensive, by weight ratio of:

-100 mesh Nat. Anhydrite : -150 mesh Fly ash: Accelerator : Water

80 : 20 : 1.2 : 20

Density = 2252 kb/m^3, after 28 days of curing
U.C.S. = 11.1 MPa
E = 1.2 GPa.

7 - FLASHPAK - The most expensive, by % of ratio of:

-100 mesh Flyash	:	Ordinary Portland Cement	:	Accelerator	:	Bentonite	:	Water
50.2	:	13.9	:	4.5	:	1	:	20.4

Density = 1558 kq/m^3, after 28 days of curing.
U.C.S. = 8 - 11 MPa
E = 0.8 - 1.1 GPa.

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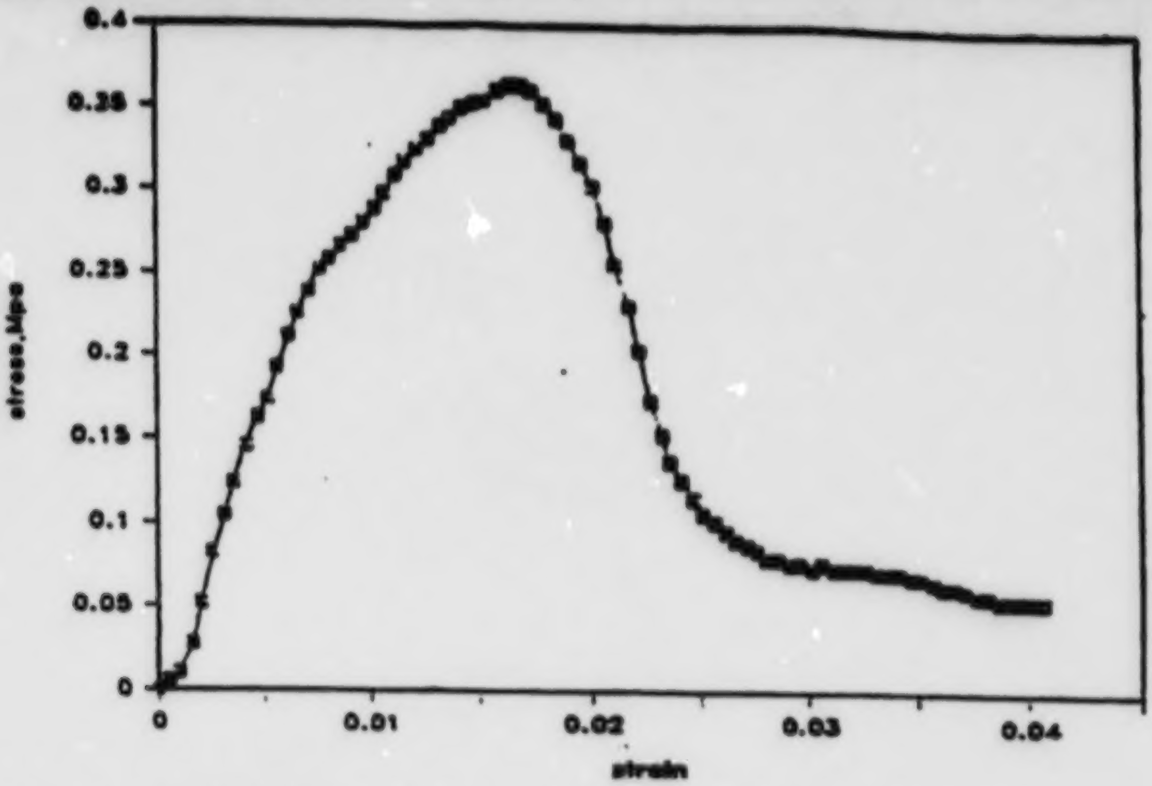
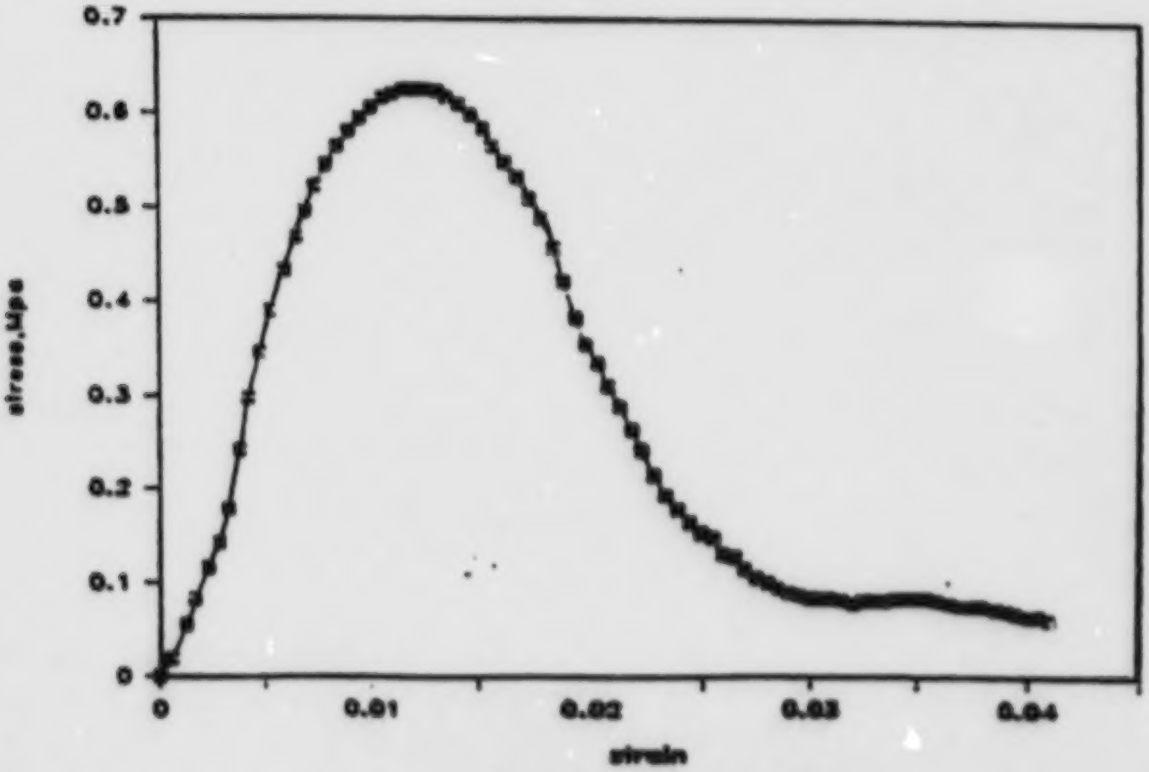
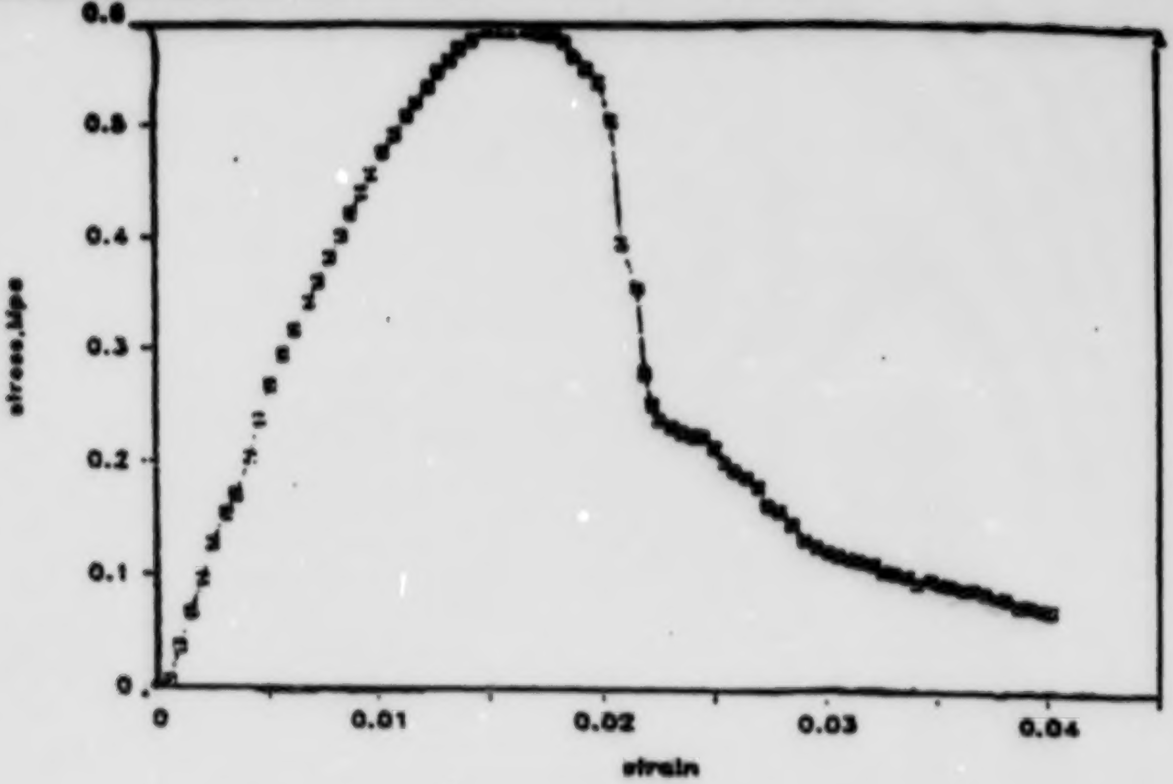
APPENDICES



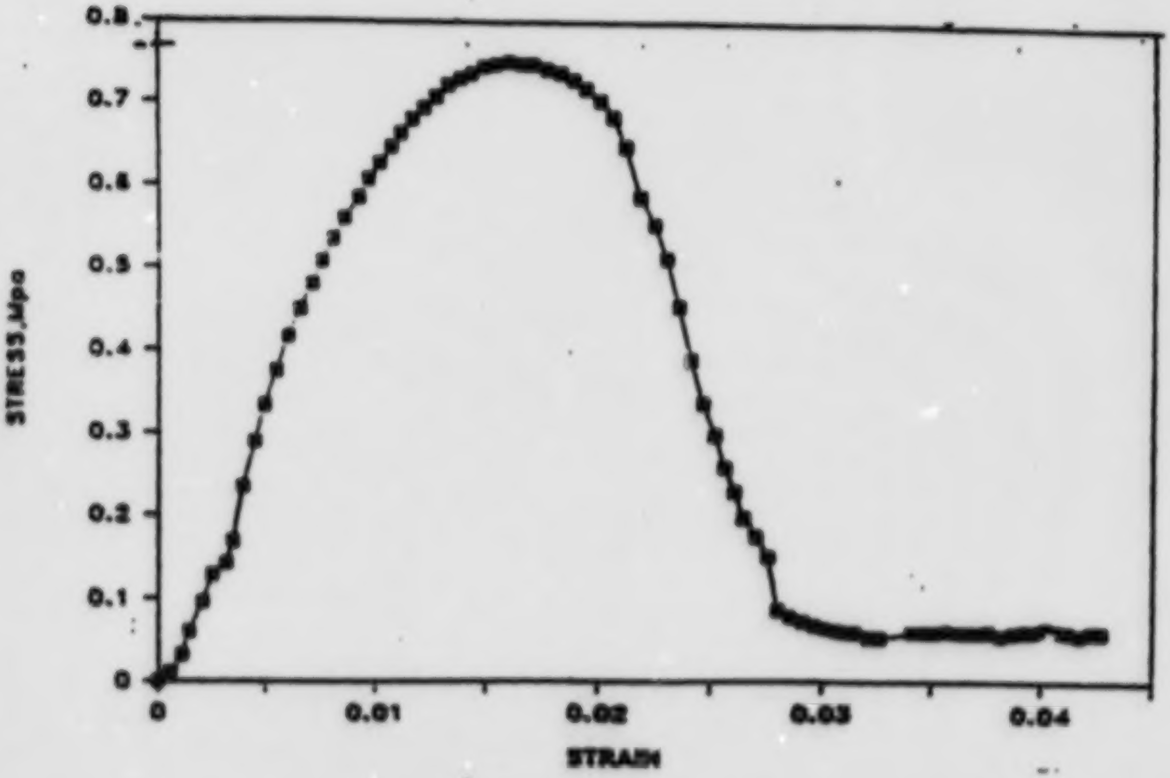
A1 - Typical stress/strain curve for ANFL-M size#1, 1 day old



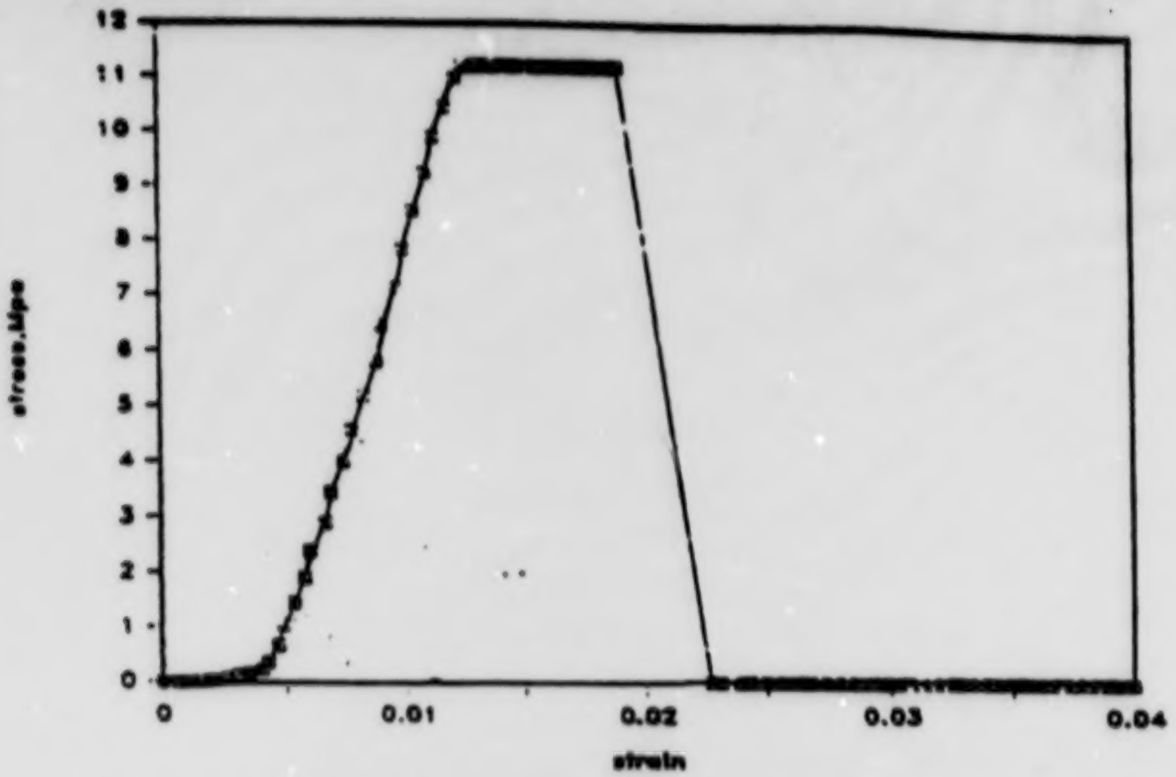
A 2 - Typical stress/strain curve for ANFL-M size#1, 3 days old



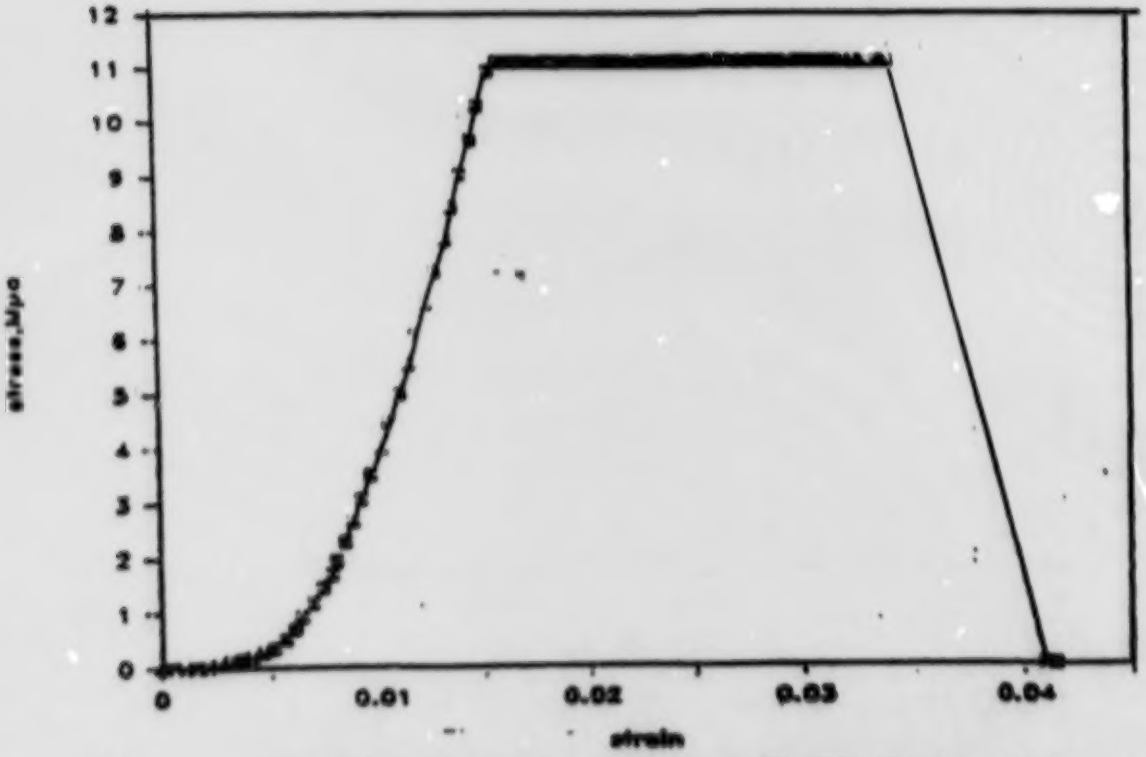
A3 - Typical stress/strain curve for ANFL-M size#1, 7 days old



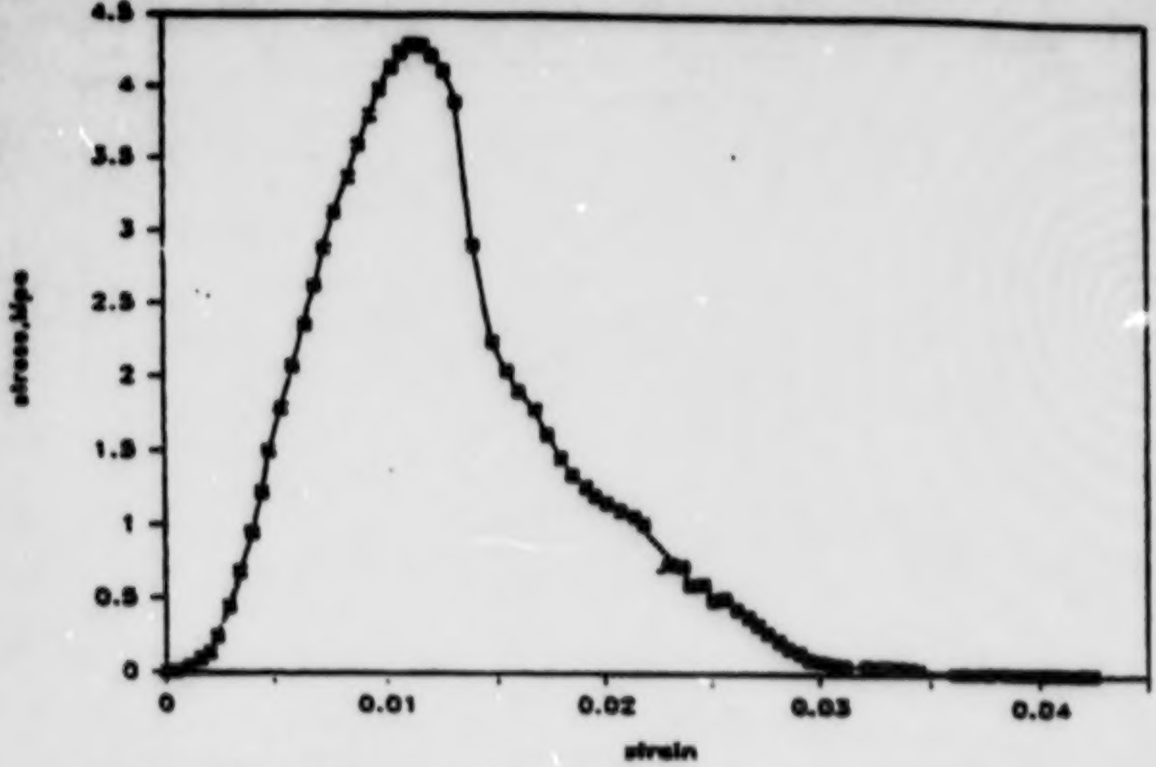
A4 - Typical stress/strain curve for ANFL-M size#1, 28 days old



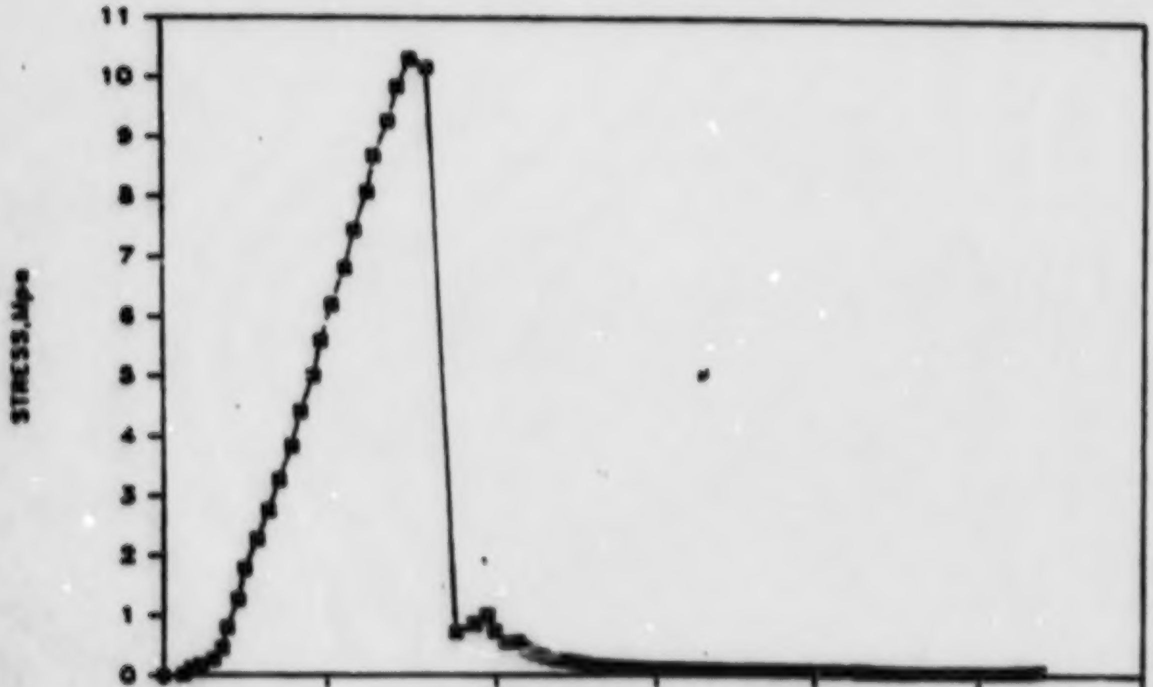
A5 - Typical stress/strain curve for ANPACK , 3 days old



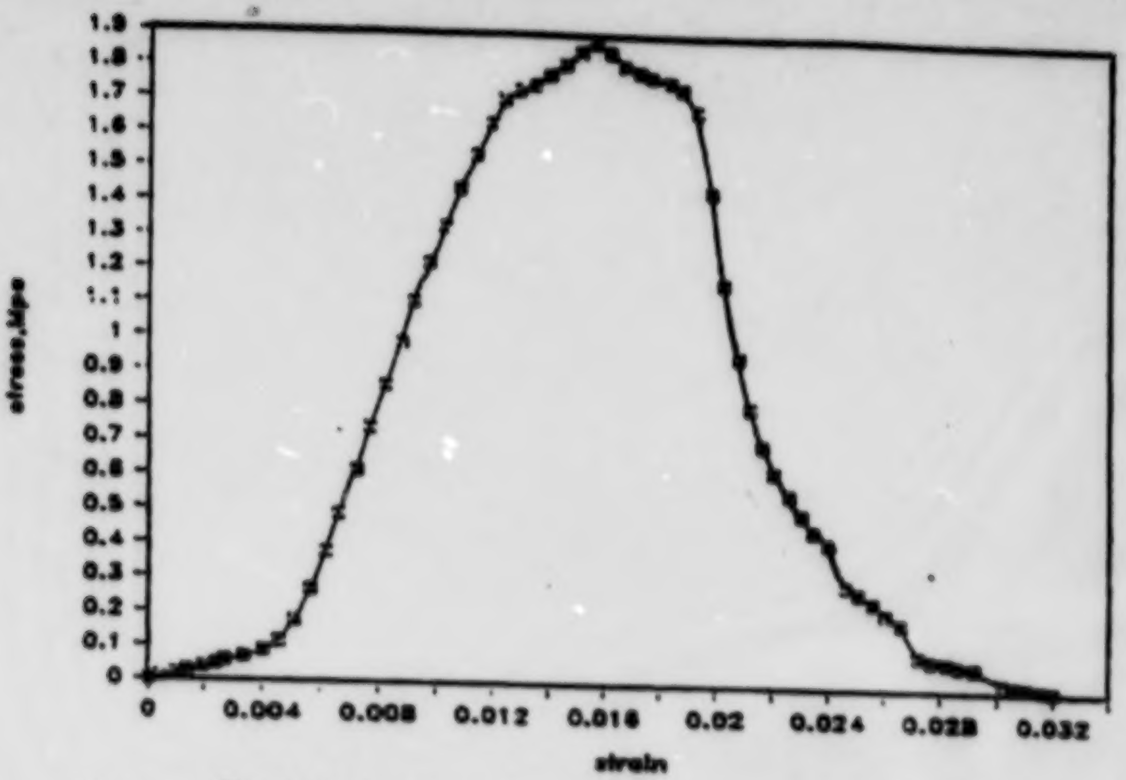
A6 - Typical stress/strain curve for ANPACK, 28 days old



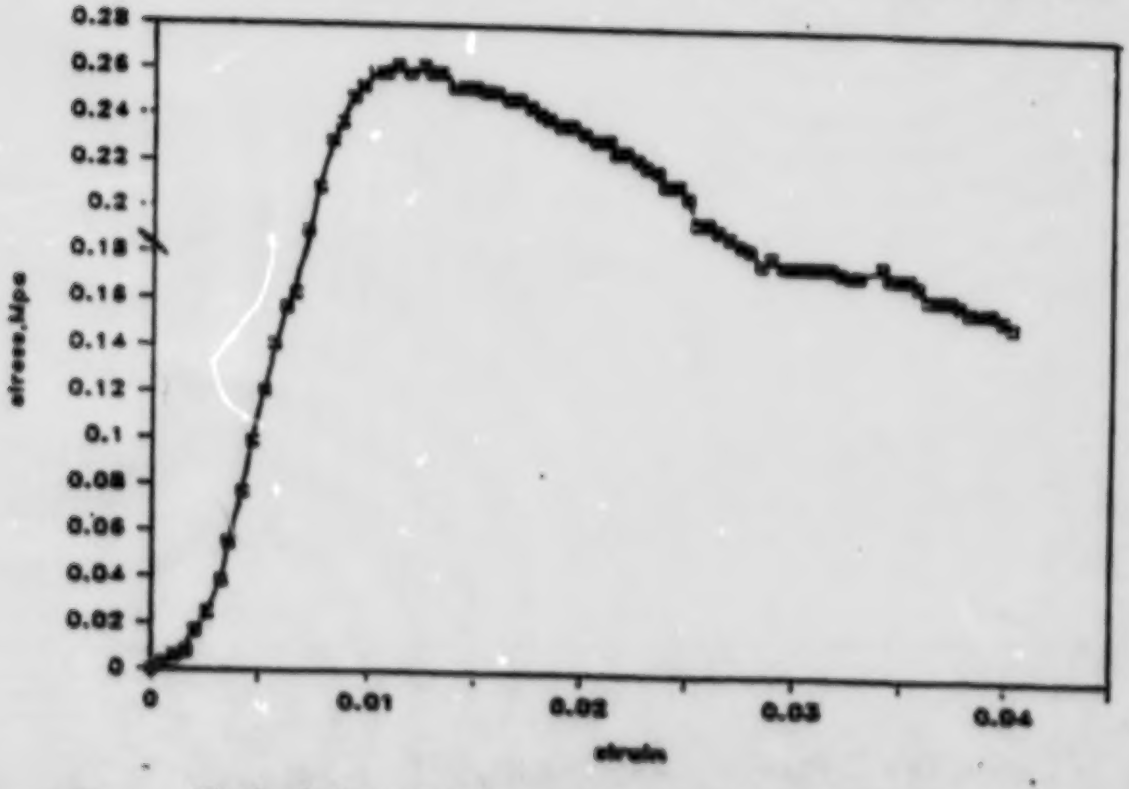
A7 - Typical stress/strain curve for PUMP PACK#2 ,7 days old



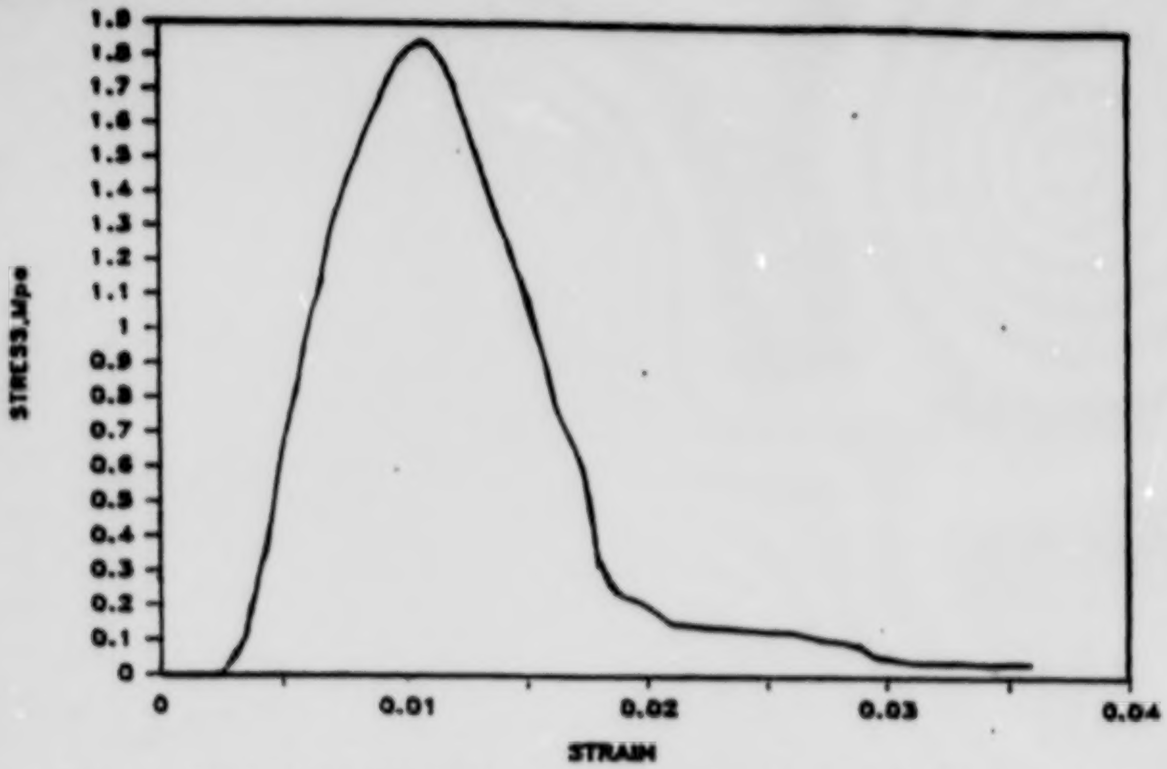
A 8 - Typical stress/strain curve for ANPACK, 1 day old



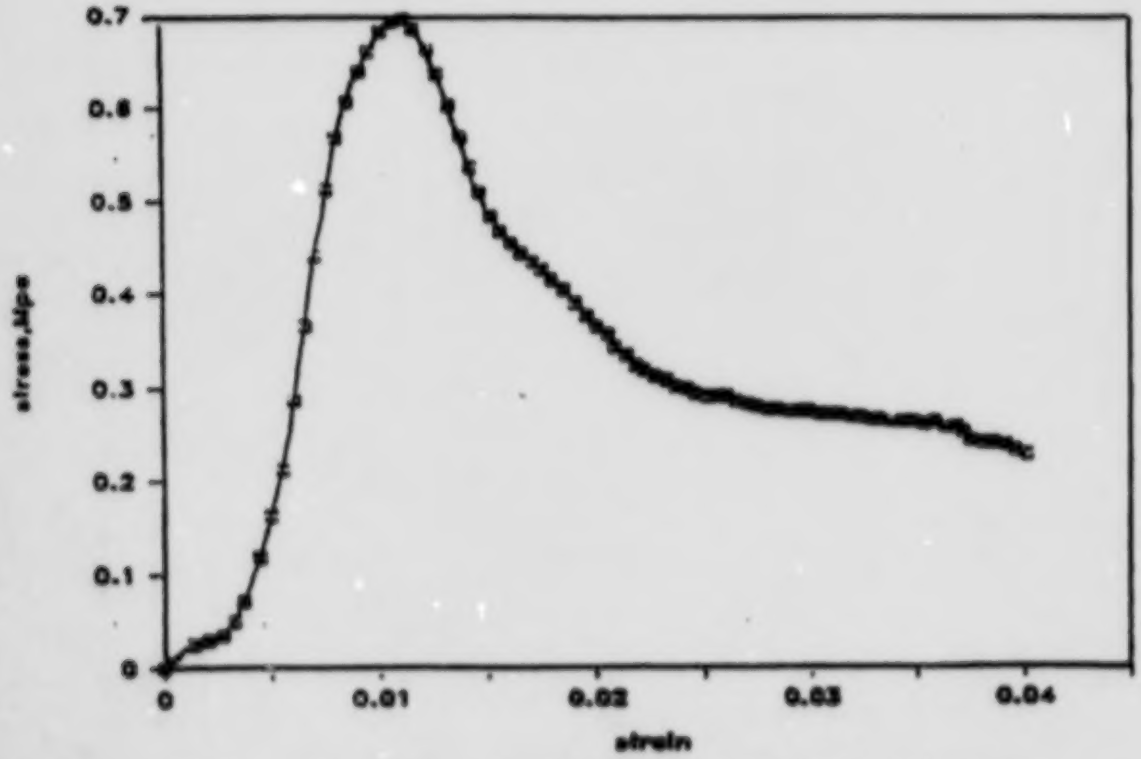
A9 - Typical stress/strain curve for PUMP PACK#1, 7 days old



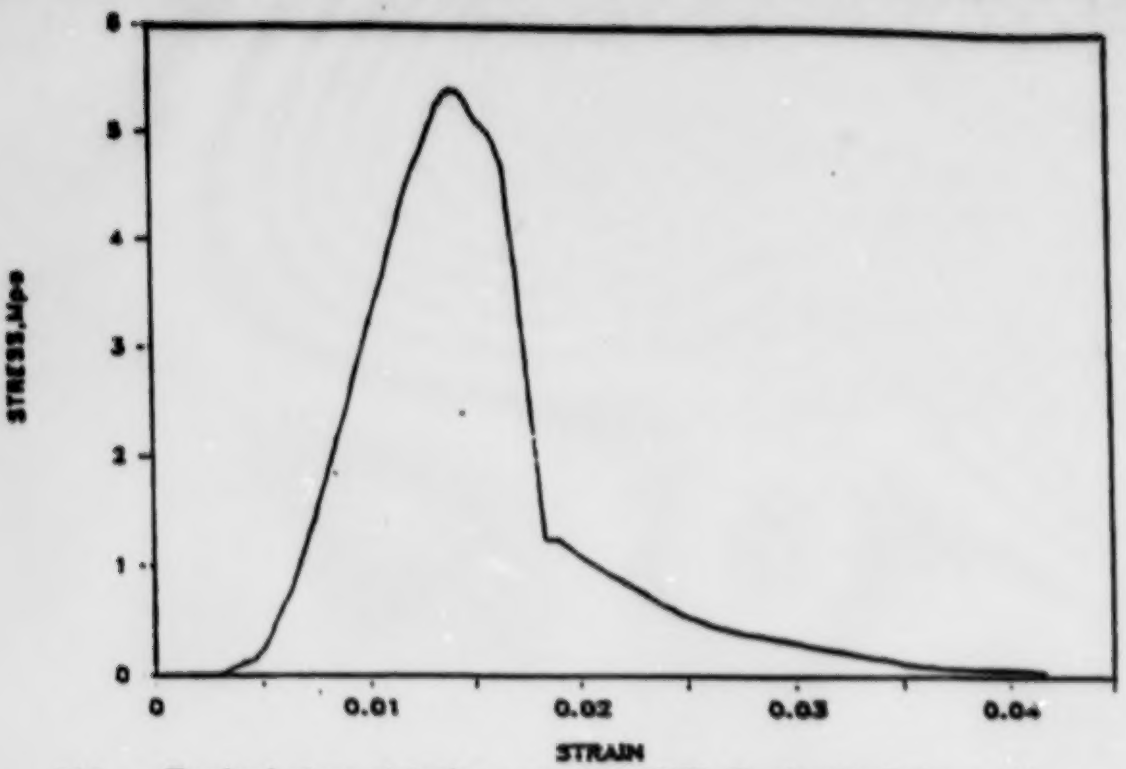
A10 - Typical stress/strain curve for PUMP PACK#2, 3 days old



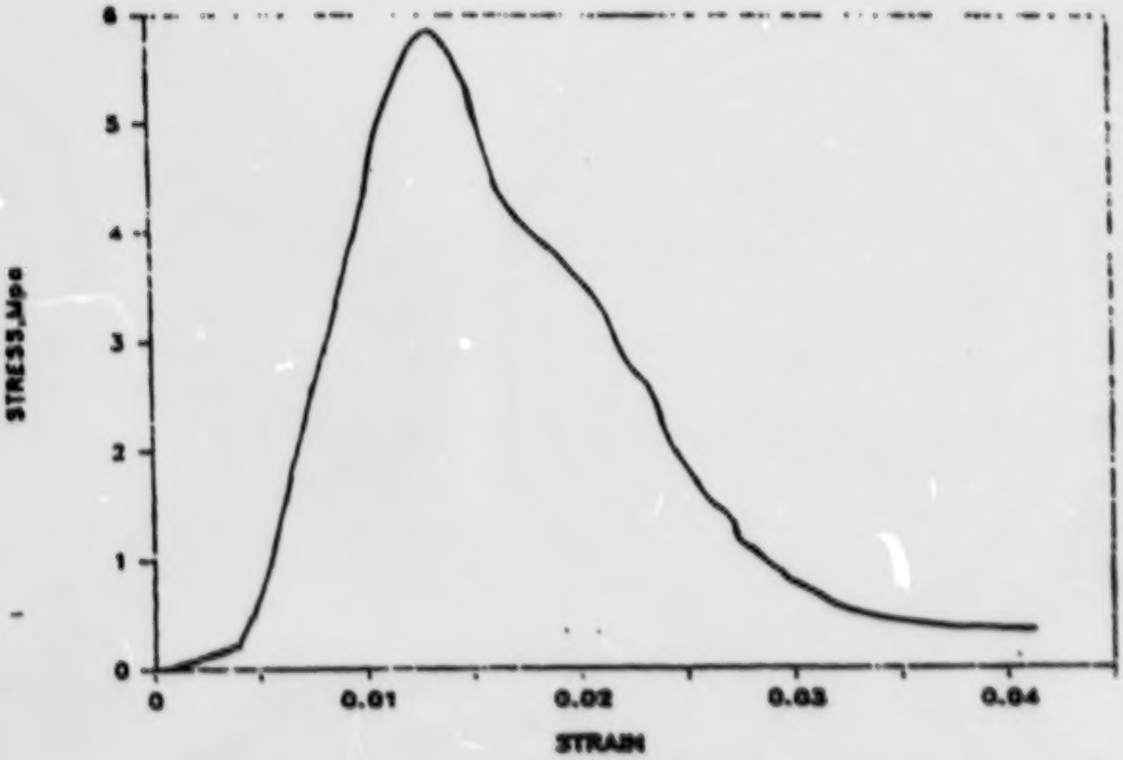
A11 - Typical stress/strain curve for ANFL-N size#1, 28 days old



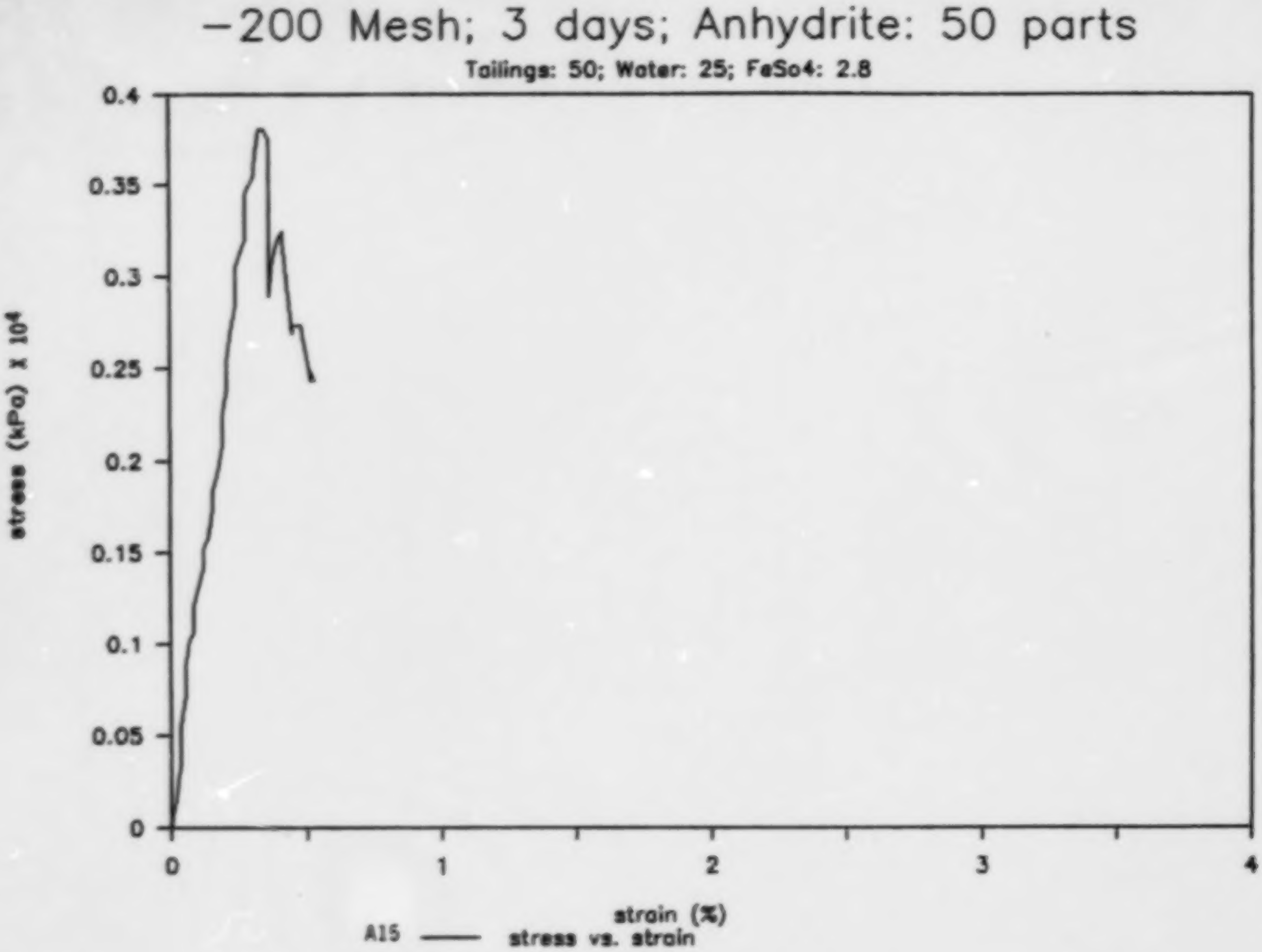
A12 - Typical stress/strain curve for PUMP PACK #1, 3 days old



A13 - Typical stress/strain curve for ANFL-P size#1, 7 days old

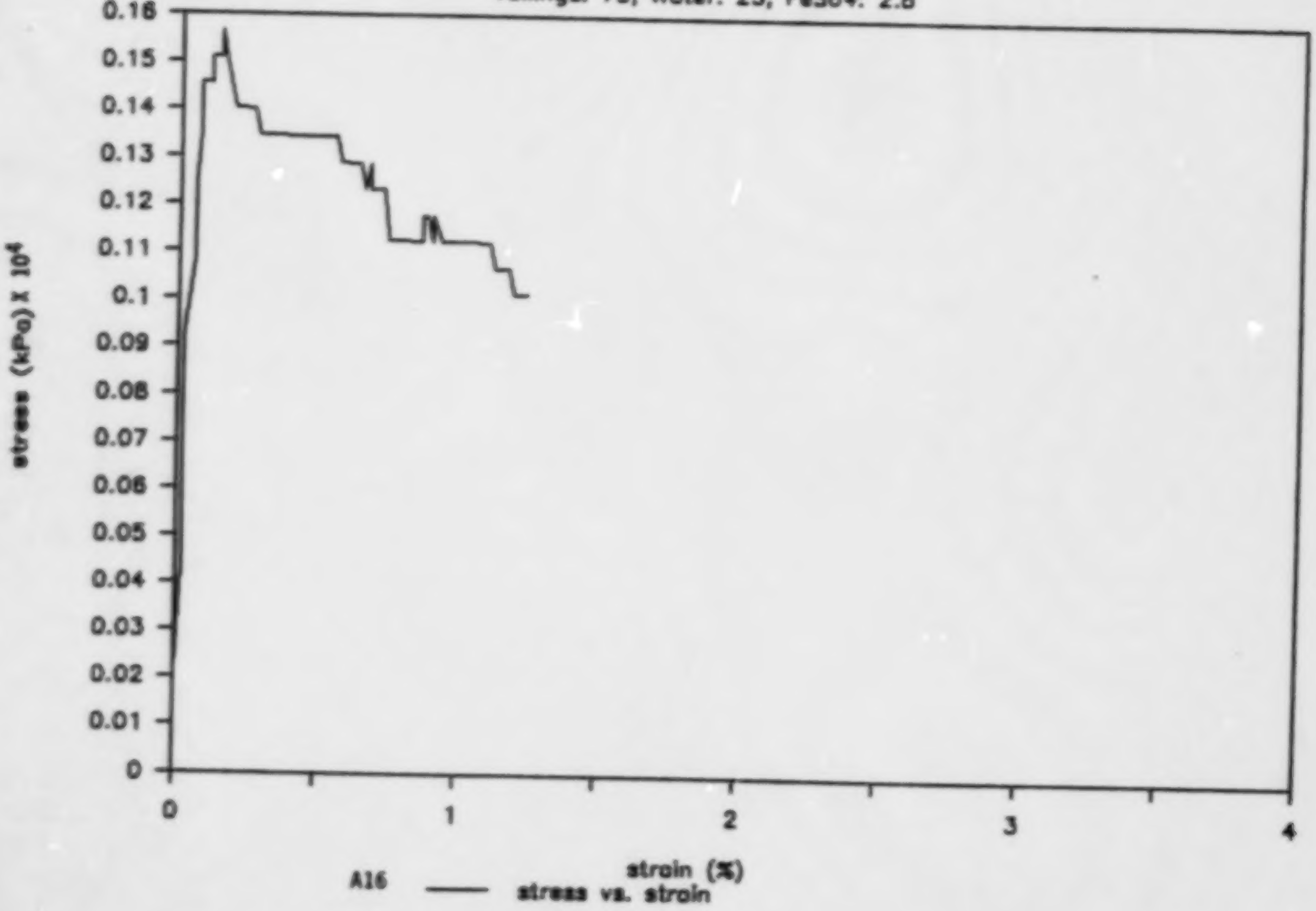


A14 - Typical stress/strain curve for ANFL-P size#1, 28 days old



−200 Mesh; 3 days; Anhydrite: 50 parts

Tailings: 50; Water: 25; FeSo4: 2.8

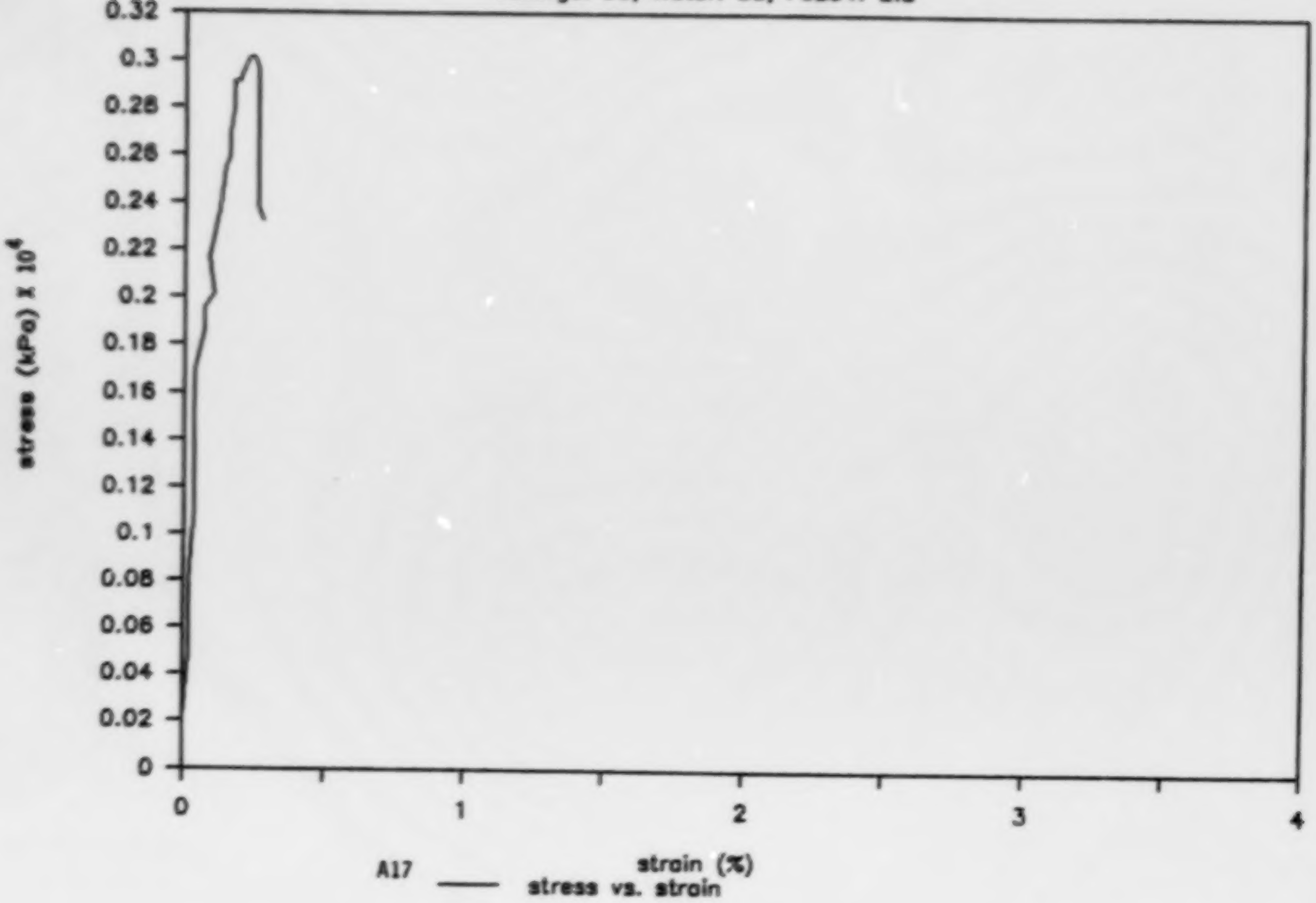


−200 Mesh; 3 days; Anhydrite: 25 parts

Tailings: 75; Water: 25; FeSo4: 2.8

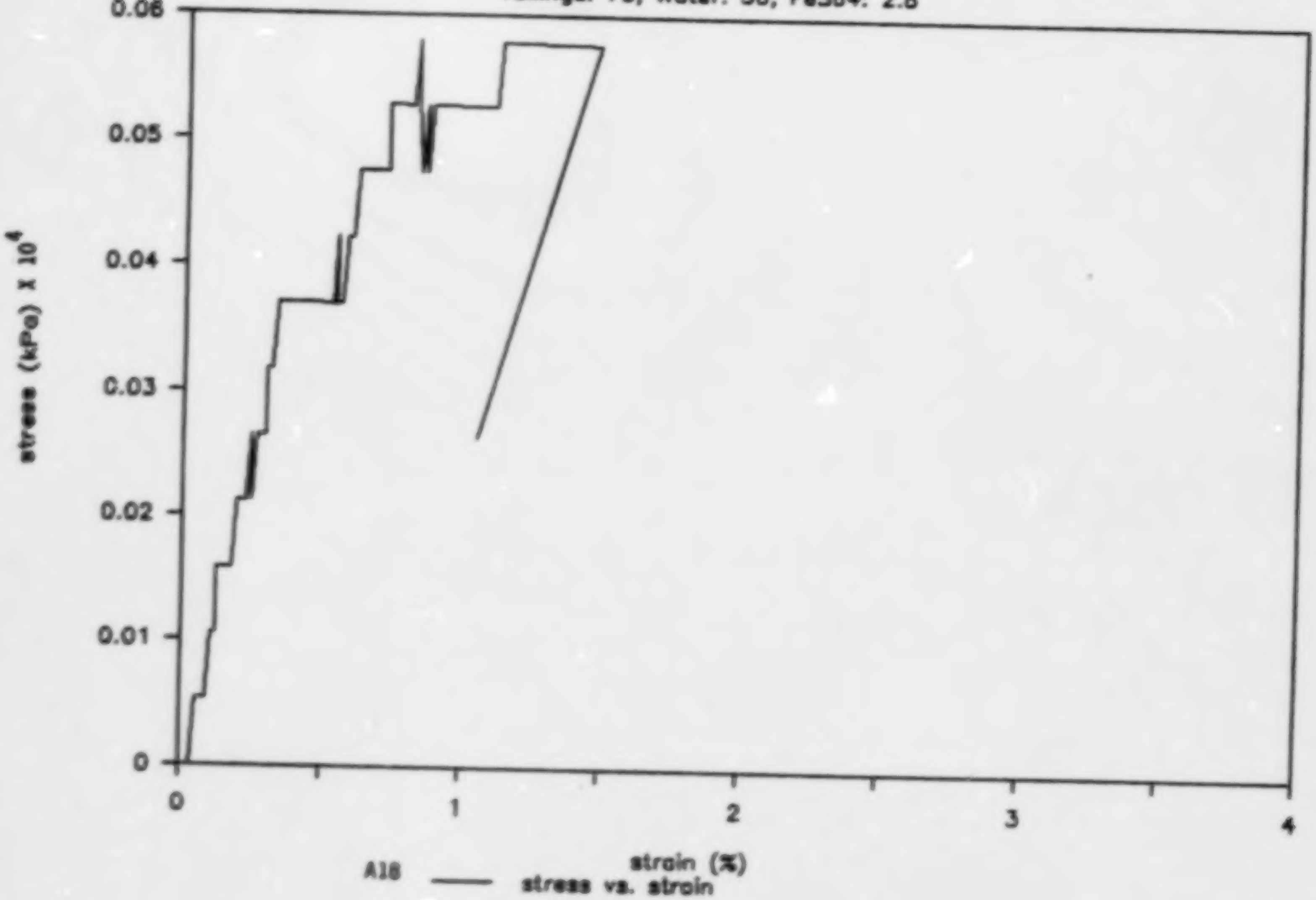
-200 Mesh; 3 days; Anhydrite: 50 parts

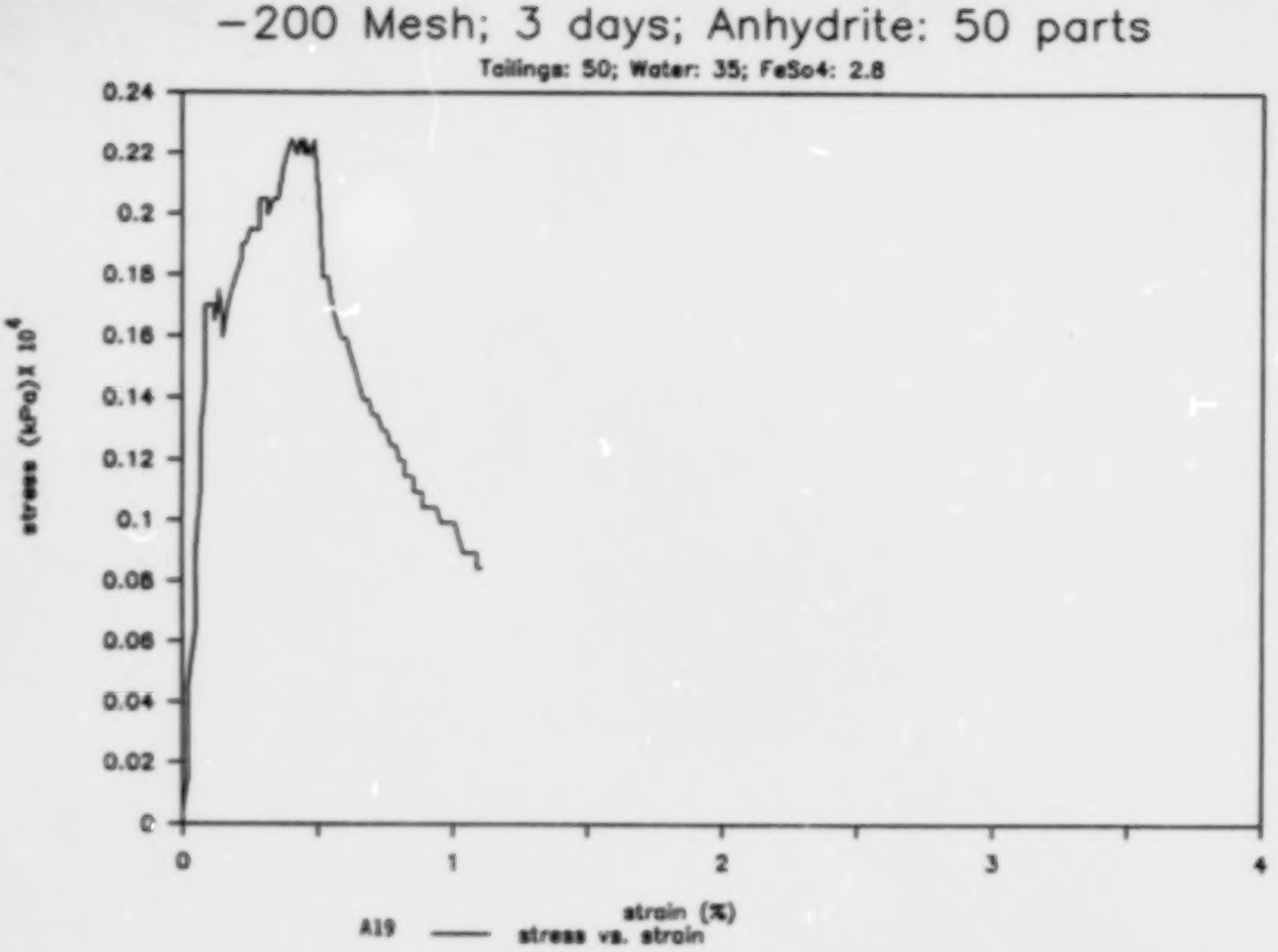
Tailings: 50; Water: 30; FeSo4: 2.8

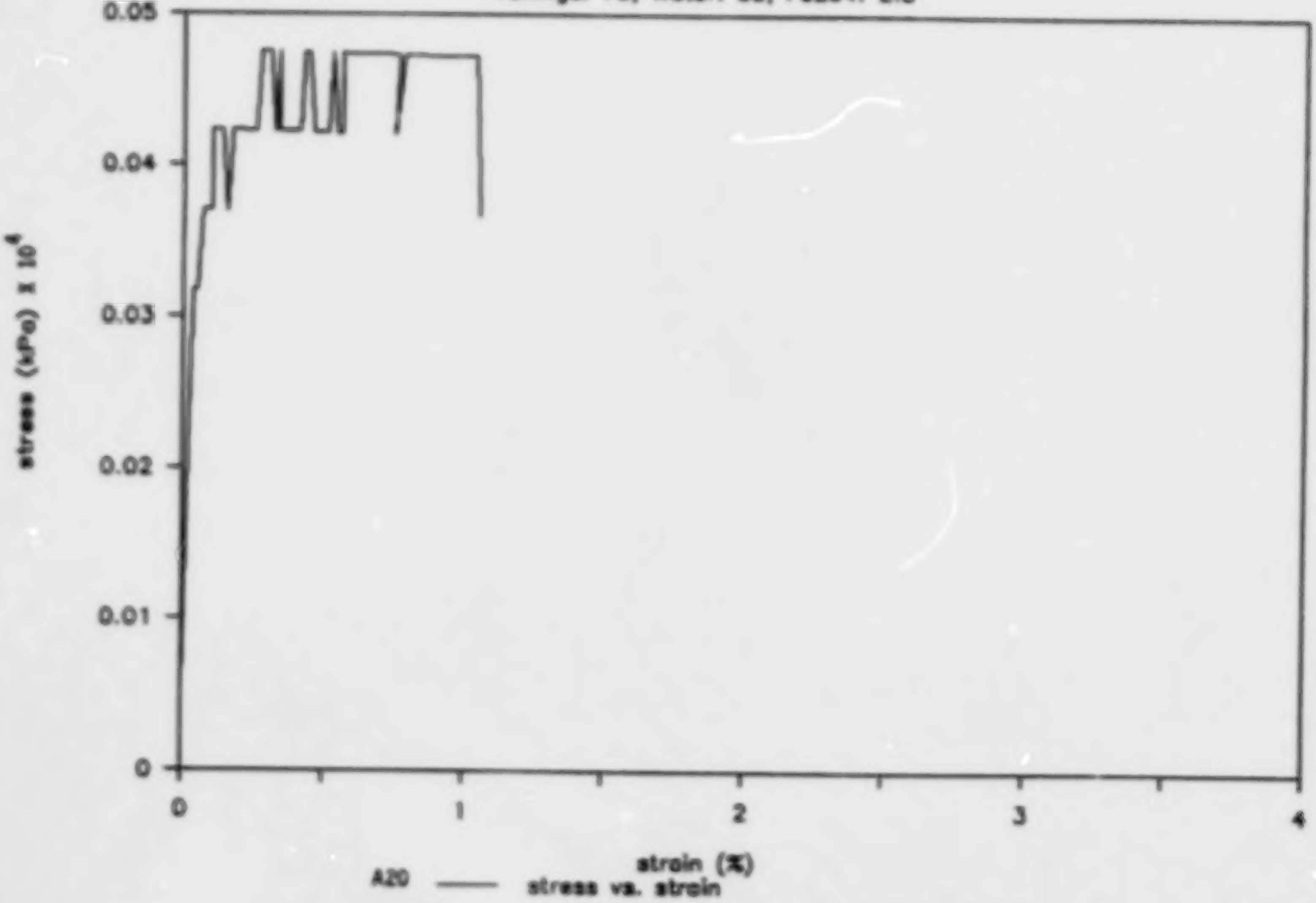


−200 Mesh; 3 days; Anhydrite: 25 parts

Tailings: 75; Water: 30; FeSo4: 2.8

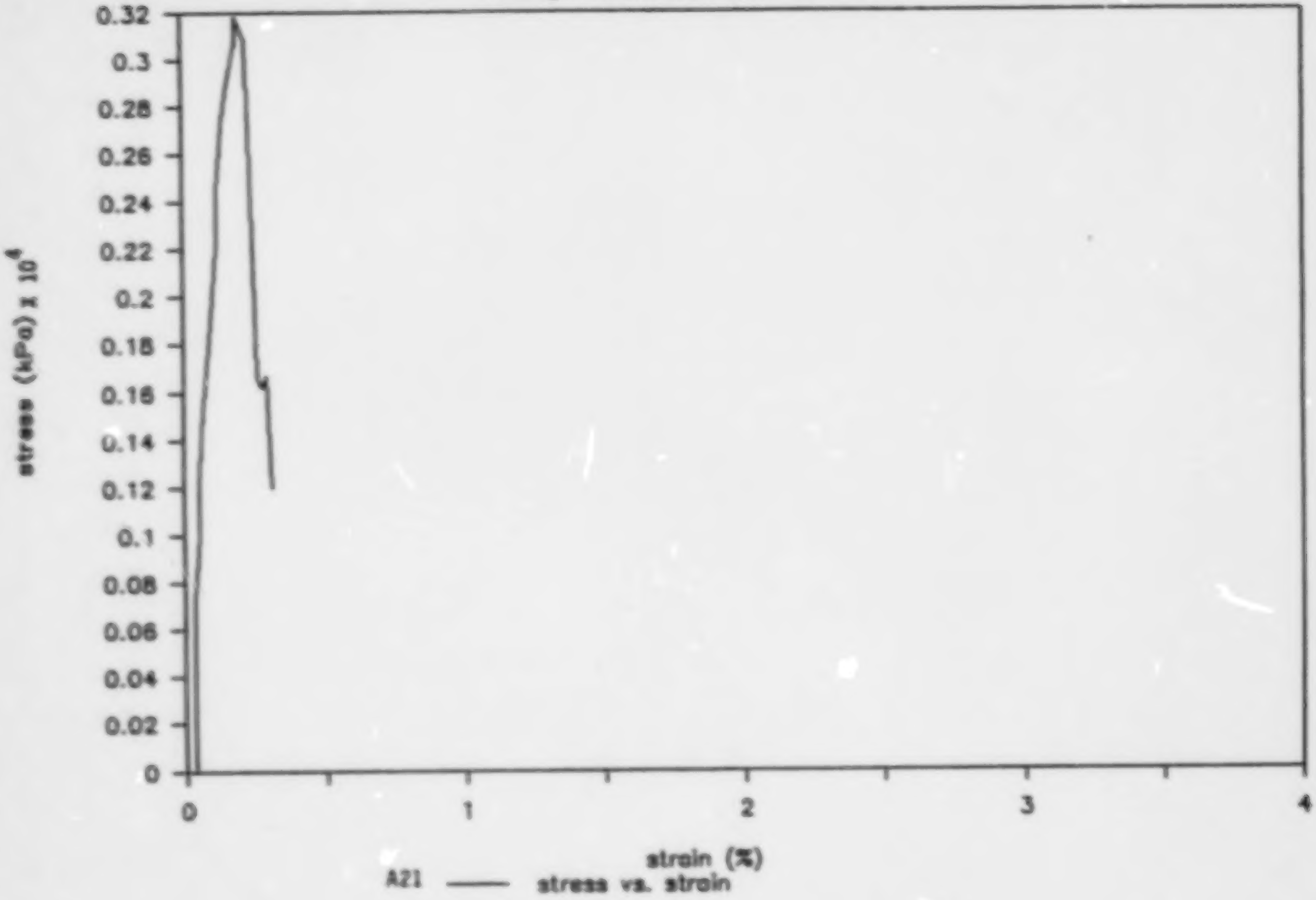



−200 Mesh; 3 days; Anhydrite: 50 parts
Tailings: 50; Water: 35; FeSo4: 2.8
stress (kPa)X 10^4
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
3
4
strain (%)
A19
stress vs. strain



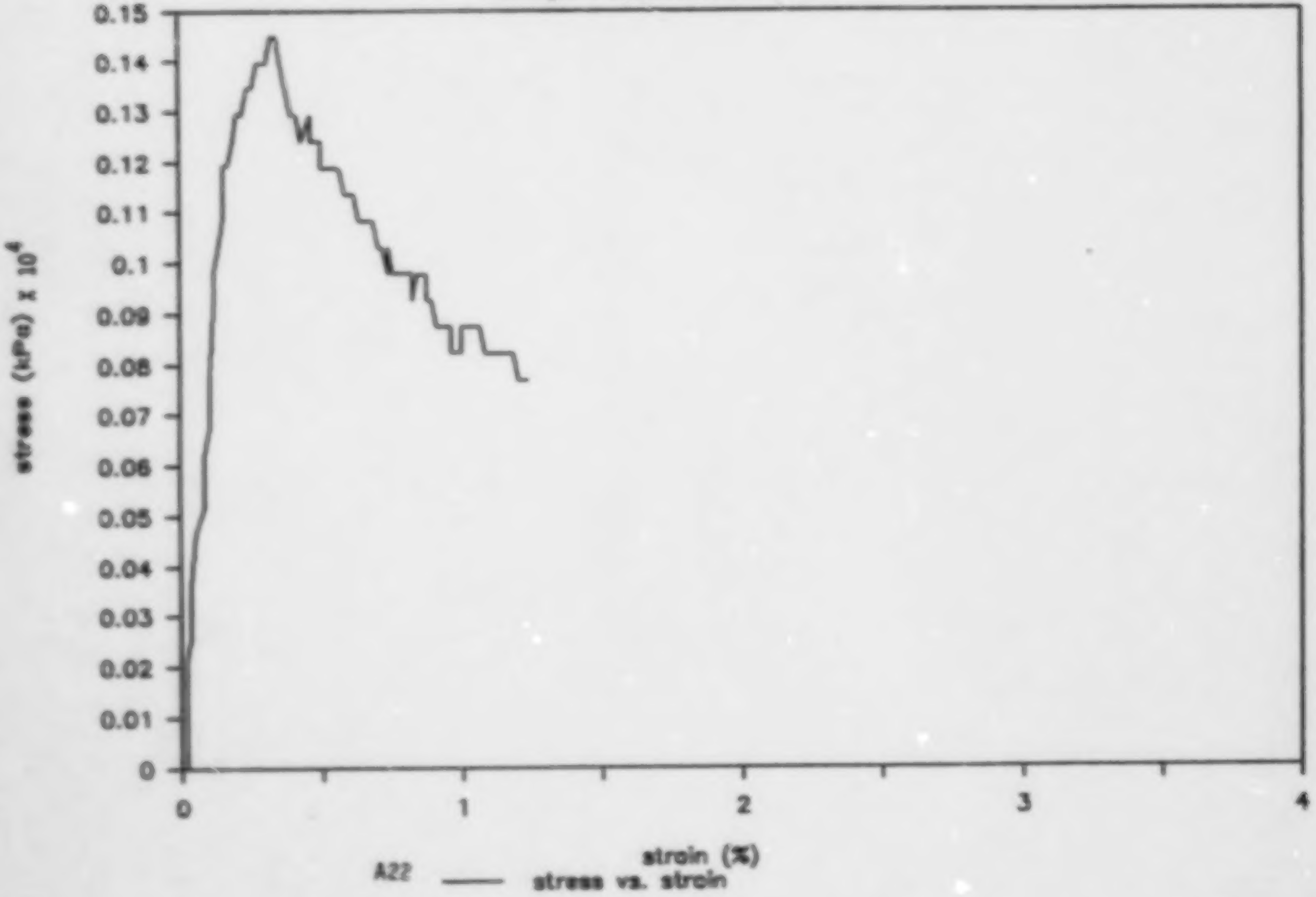
−200 Mesh; 3 days; Anhydrite: 25 parts

Tailings: 75; Water: 35; FeSo4: 2.8



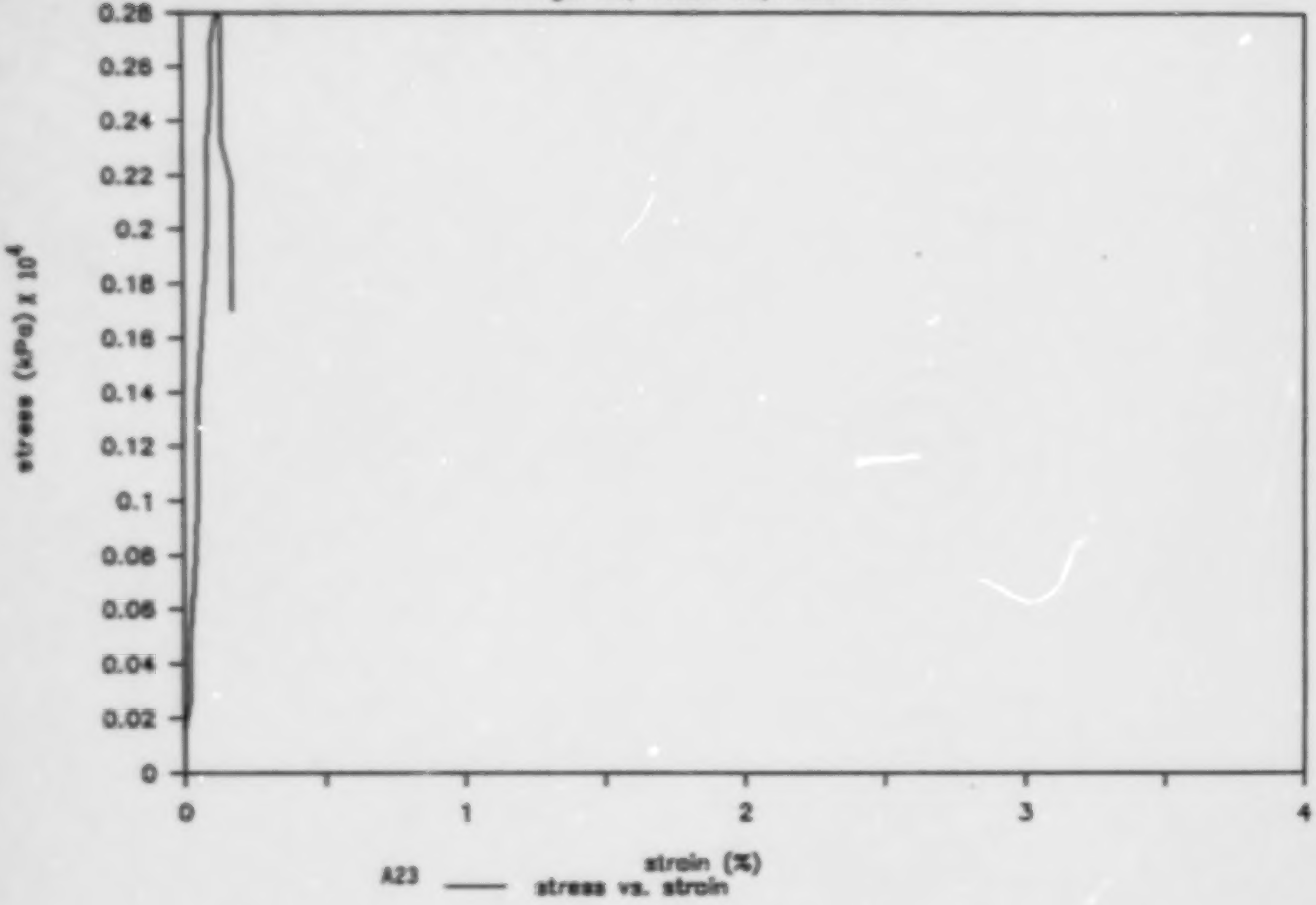
−200 Mesh; 3 days; Anhydrite: 50 parts

Tailings: 50; Water: 25; FeSo4: 3.6



−200 Mesh; 3 days; Anhydrite: 25 parts

Tailings: 75; Water: 25; FeSo4: 3.6

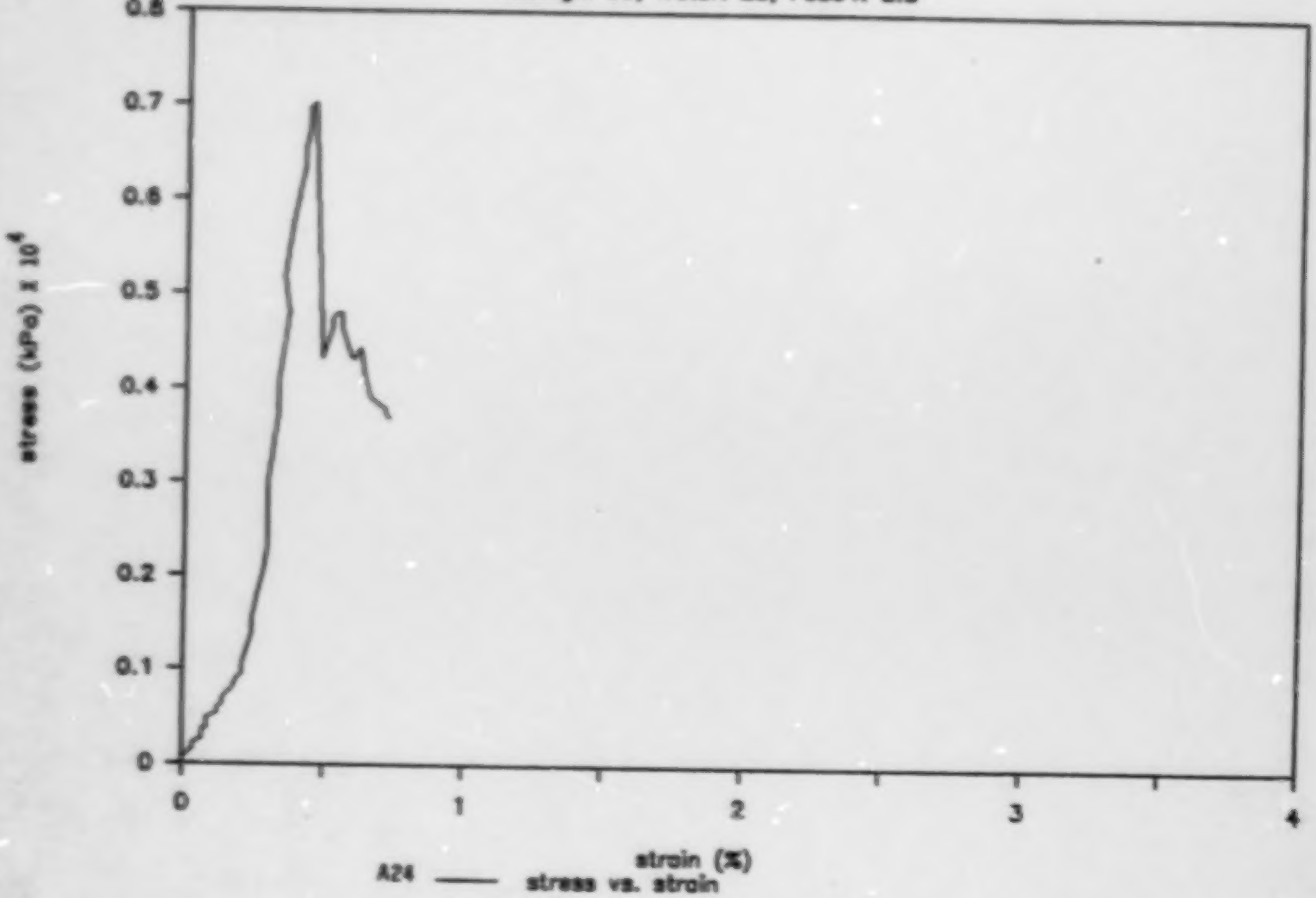


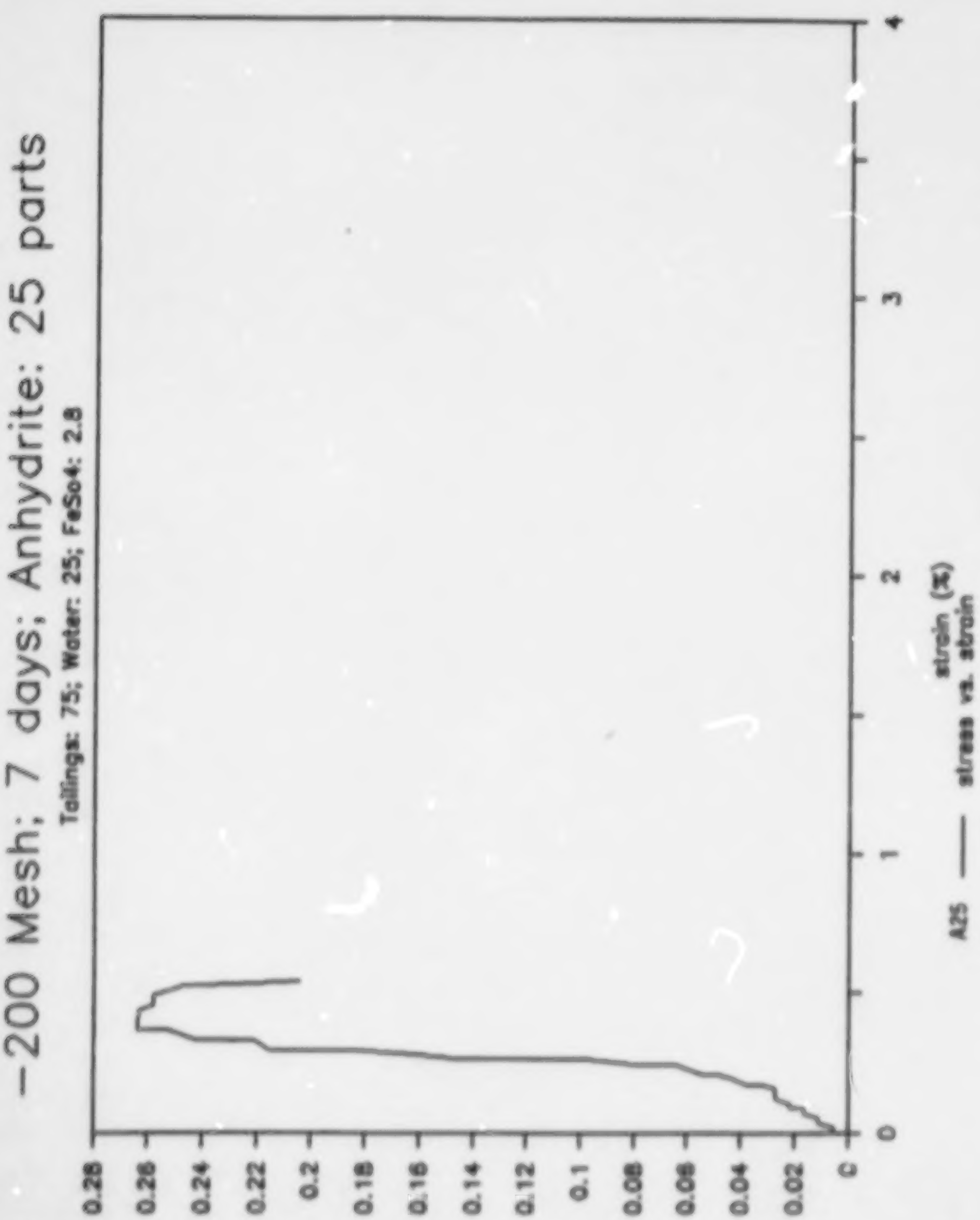
-200 Mesh; 3 days; Anhydrite: 50 parts

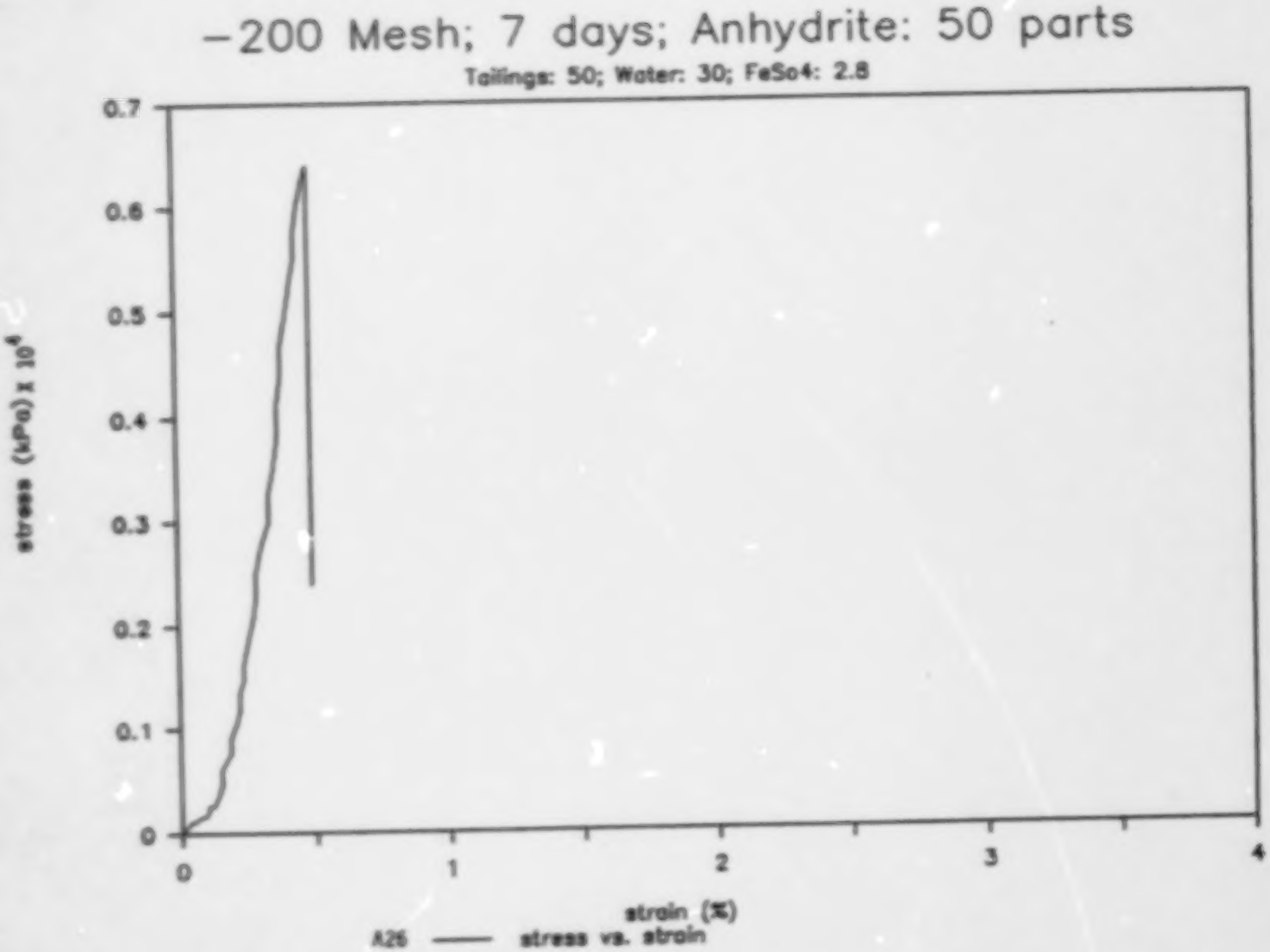
Tailings: 50; Water: 30; FeSo4: 3.6

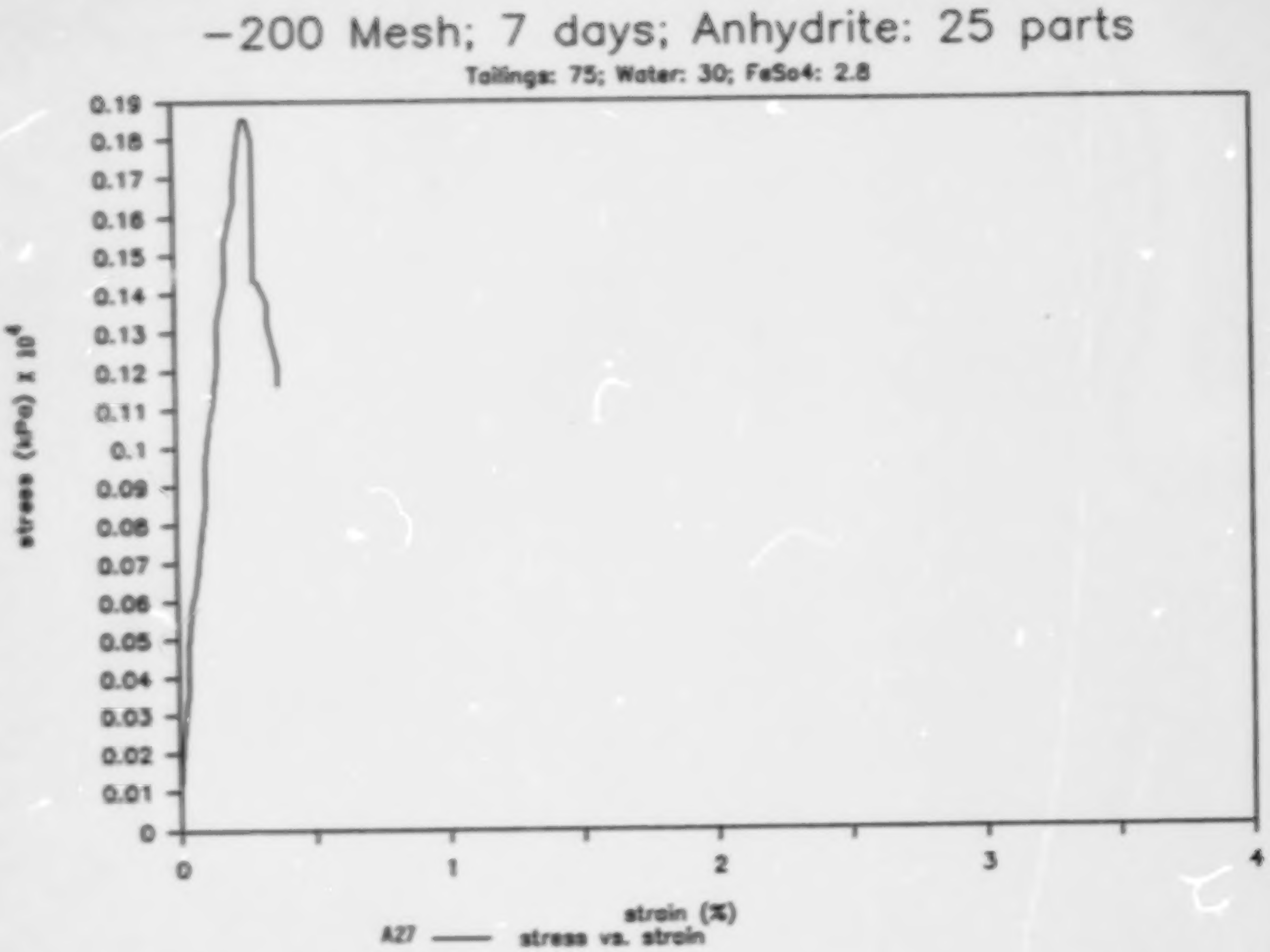
−200 Mesh; 7 days; Anhydrite: 50 parts

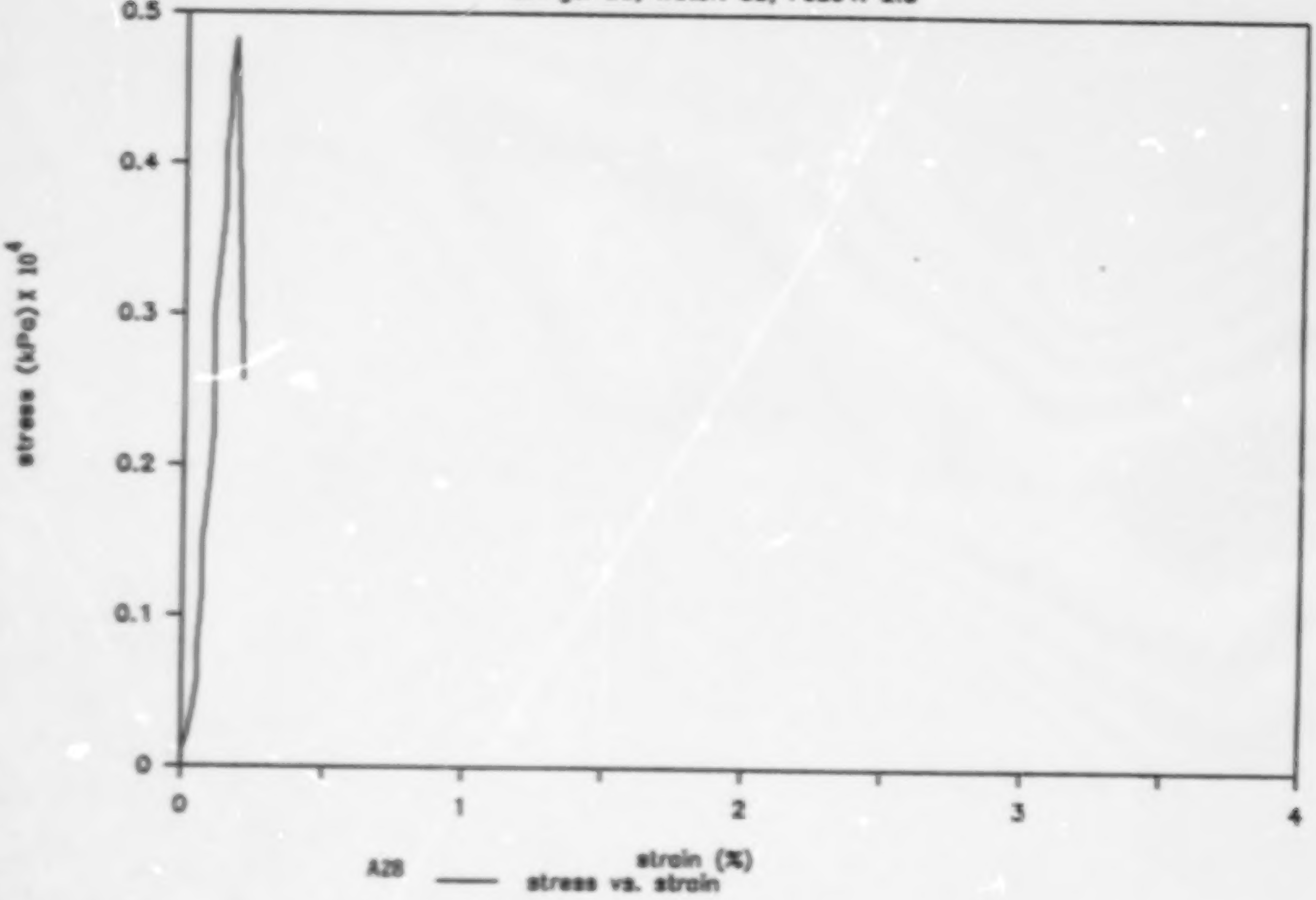
Tailings: 50; Water: 25; FeSo4: 2.8




-200 Mesh; 7 days; Anhydrite: 25 parts
Tailings: 75; Water: 25; FeSo4: 2.8
stress (kPa) X 10^4
0.28
0.26
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
3
4
strain (%)
A25 —— stress vs. strain

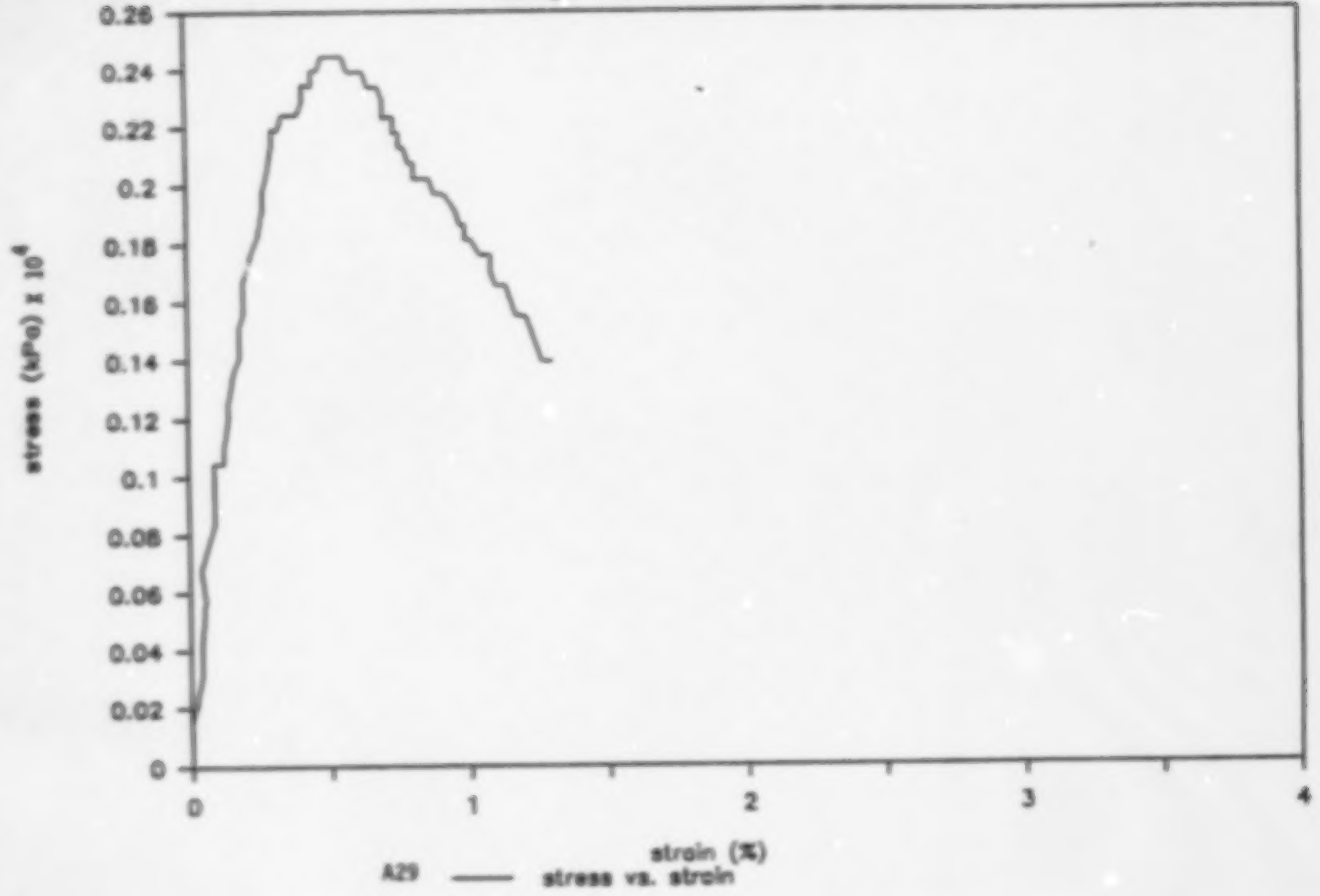

−200 Mesh; 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 30; FeSo4: 2.8
stress (kPa) x 10^4
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A26 —— stress vs. strain


-200 Mesh; 7 days; Anhydrite: 25 parts
Tailings: 75; Water: 30; FeSo4: 2.8
stress (kPa) x 10^4
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A27 —— stress vs. strain



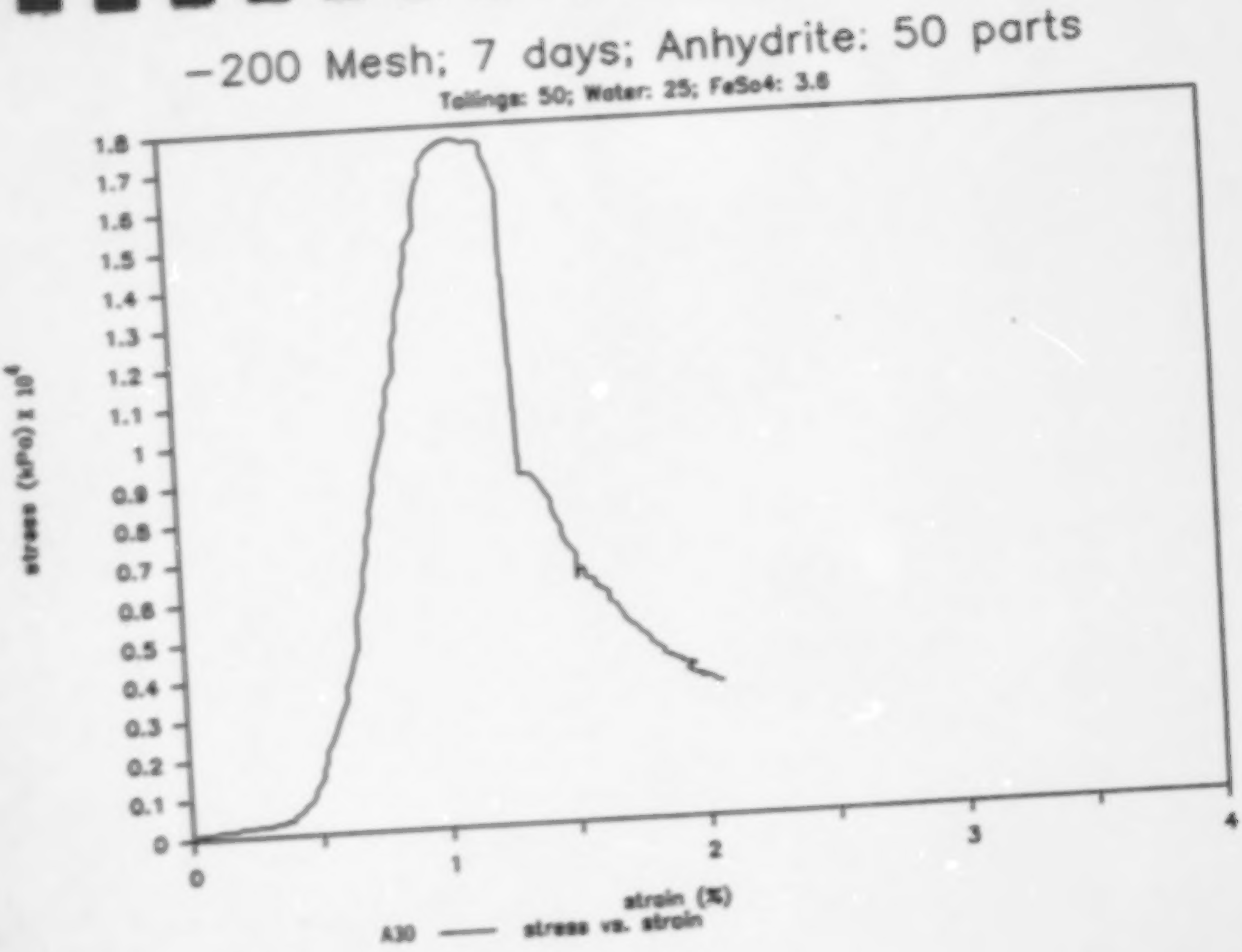
-200 Mesh; 7 days; Anhydrite: 50 parts

Tailings: 50; Water: 35; FeSo4: 2.8



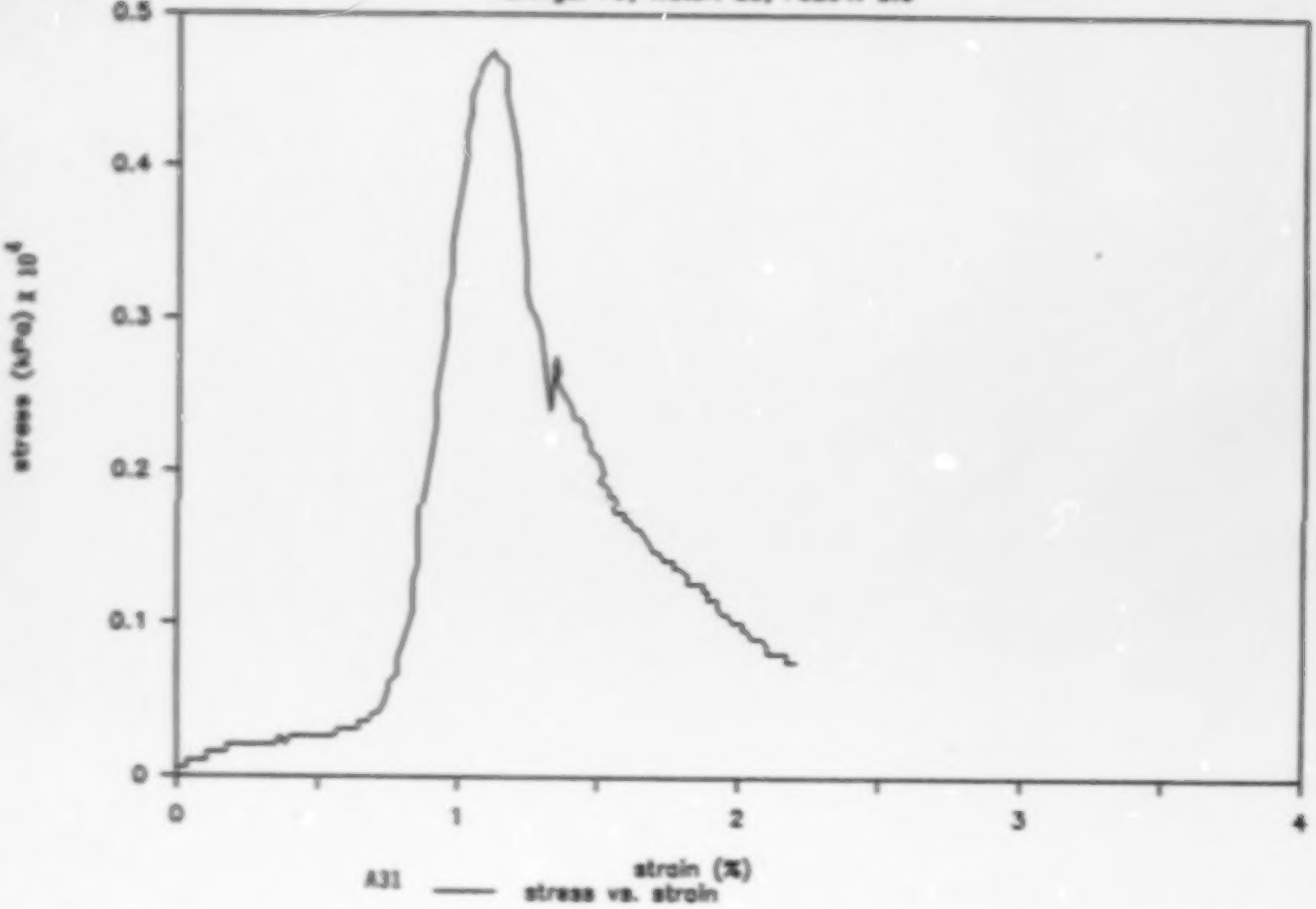
-200 Mesh; 7 days; Anhydrite: 25 parts

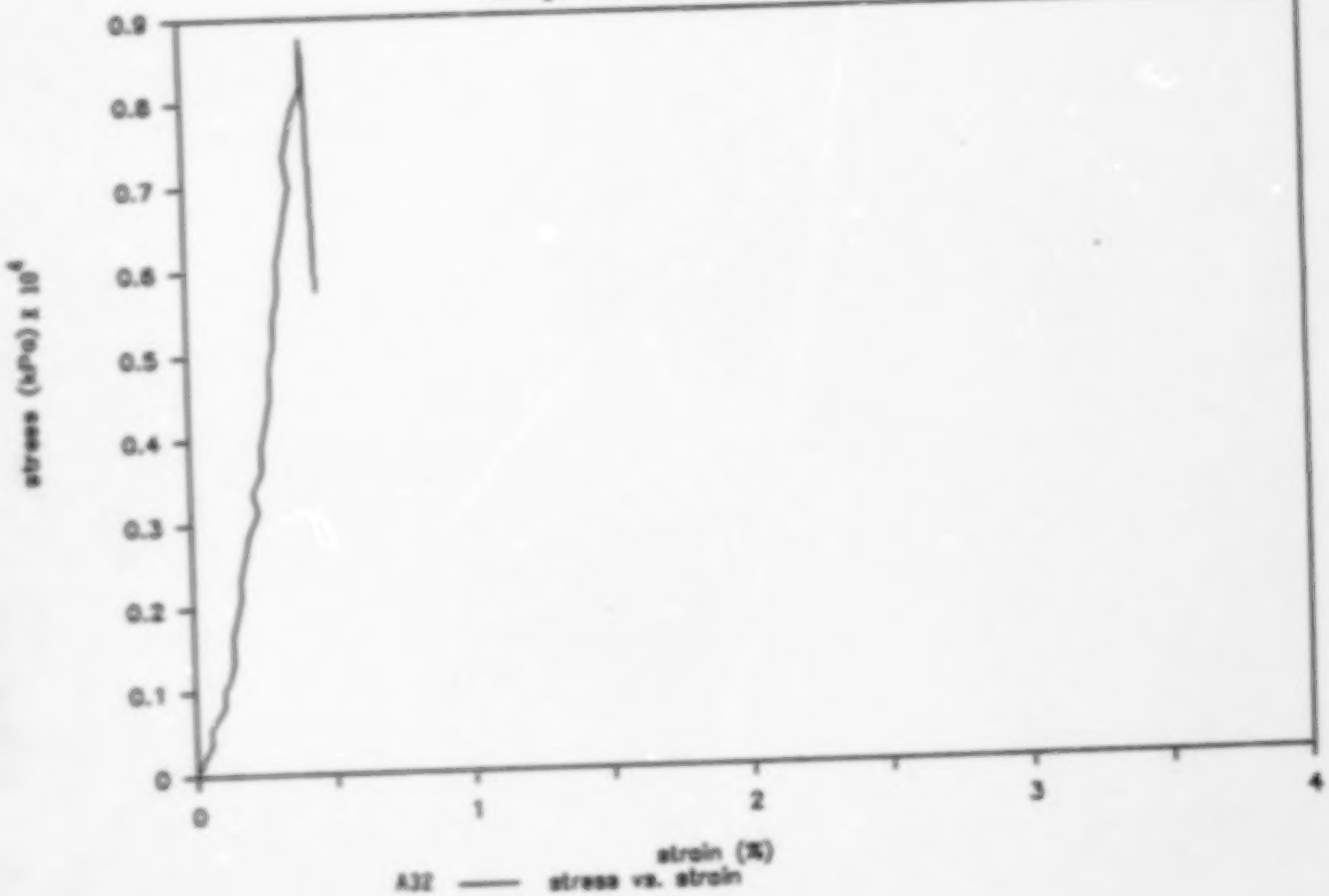
Tailings: 75; Water: 35; FeSo4: 2.8


-200 Mesh; 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 25; FeSo4: 3.6
stress (kPa) x 10^4
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A30 —— stress vs. strain

−200 Mesh; 7 days; Anhydrite: 25 parts

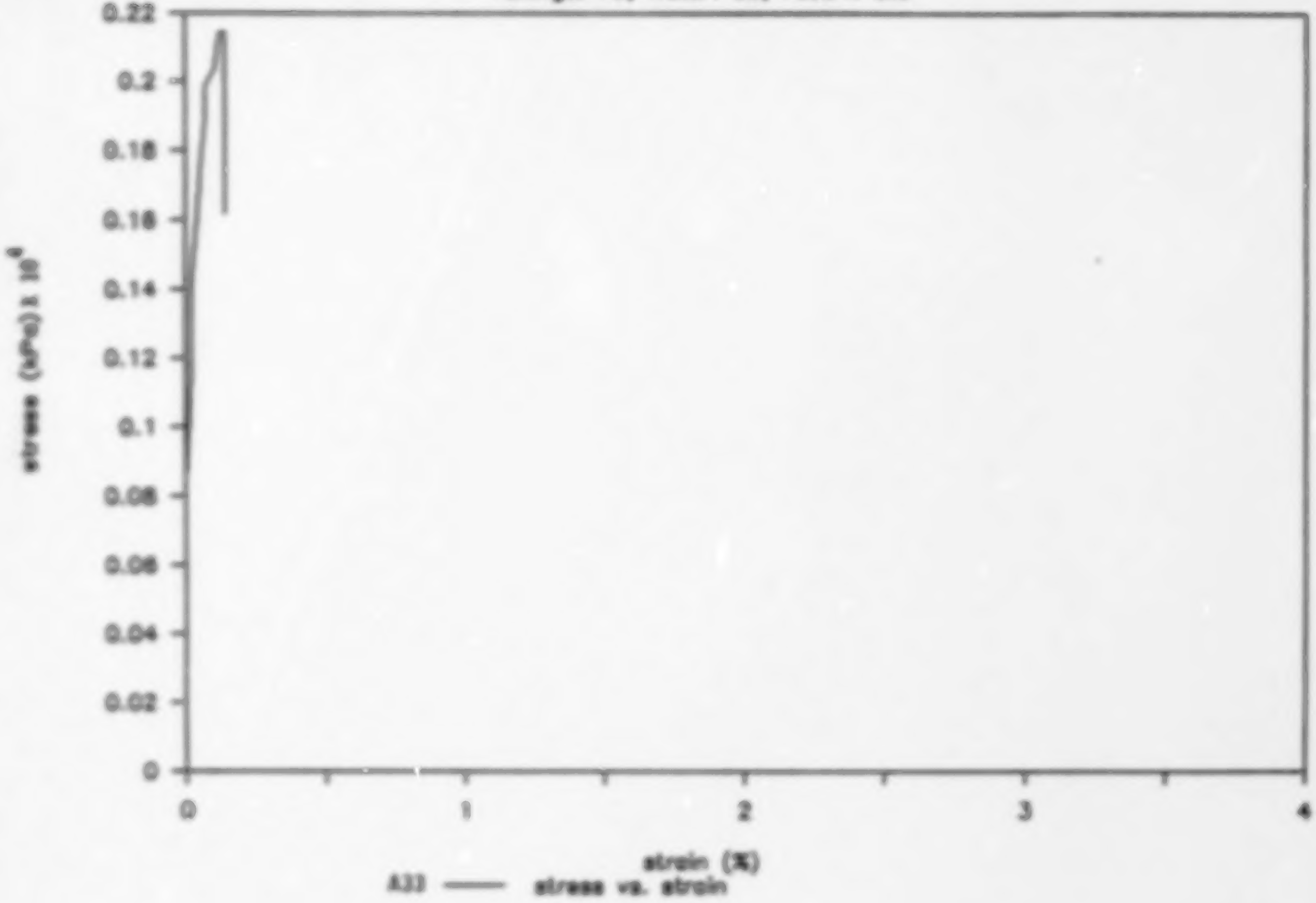
Tailings: 75; Water: 25; FeSo4: 3.6





−200 Mesh; 7 days; Anhydrite: 50 parts

Tailings: 50; Water: 30; FeSo4: 3.6

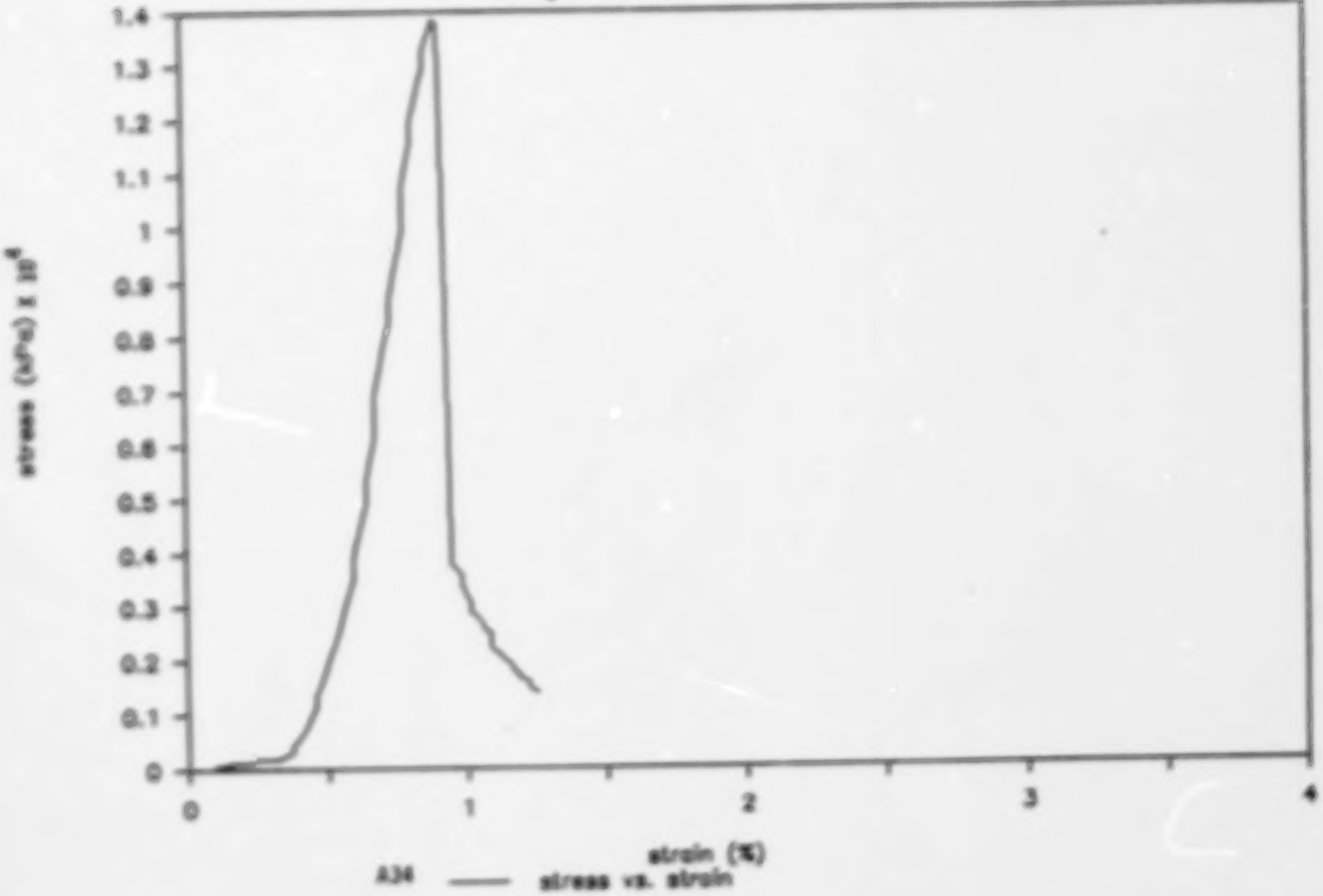


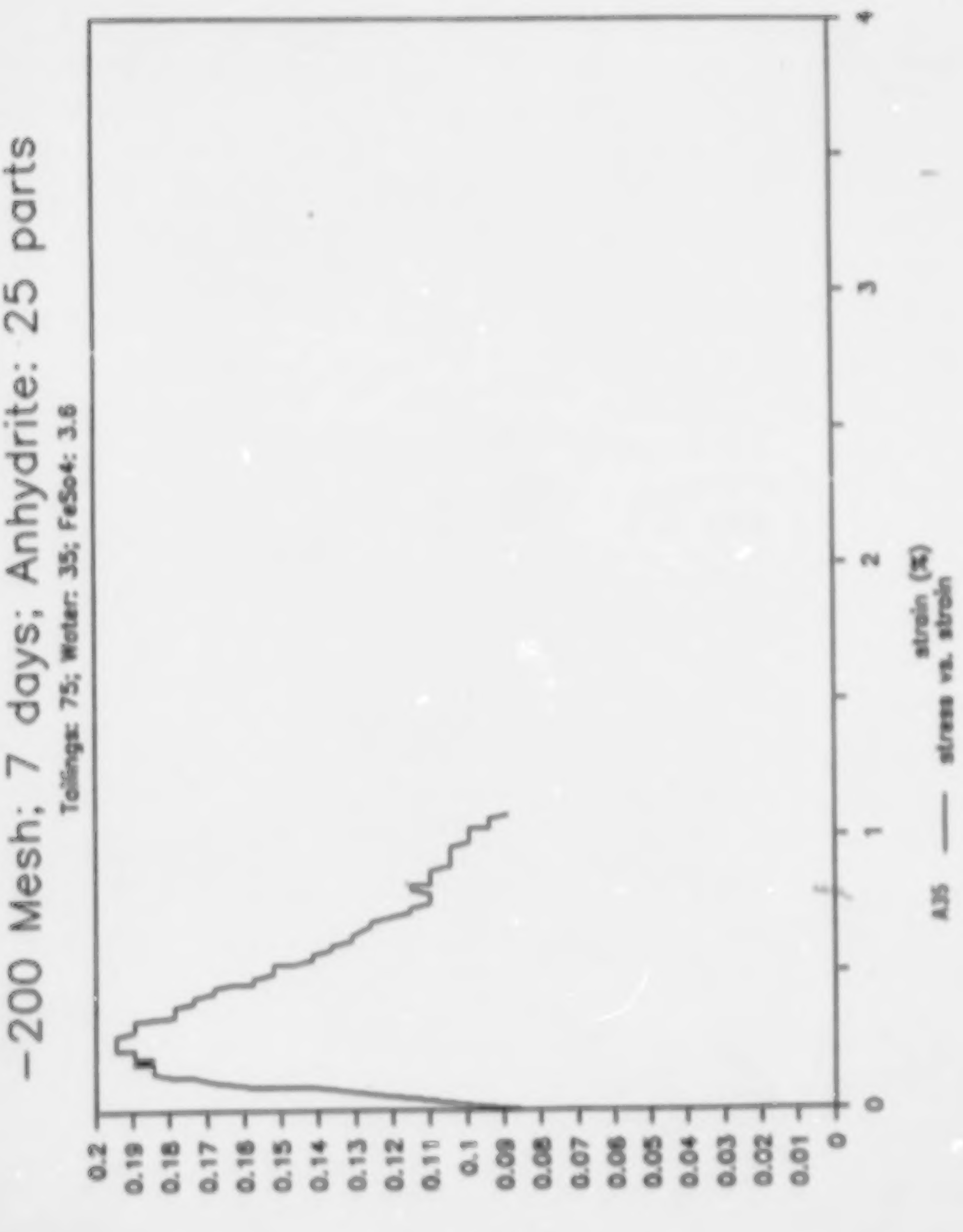
−200 Mesh; 7 days; Anhydrite: 25 parts

Tailings: 75; Water: 30; FeSo4: 3.8

−200 Mesh; 7 days; Anhydrite: 50 parts

Tailings: 50; Water: 35; FeSo4: 3.8




-200 Mesh; 7 days; Anhydrite: 25 parts
Tailings: 75; Water: 35; FeSo4: 3.6
stress (kPa) X 10^4
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A35 —— stress vs. strain

A.36

A37

$\times 10^4$

A39
X 10⁴

A40

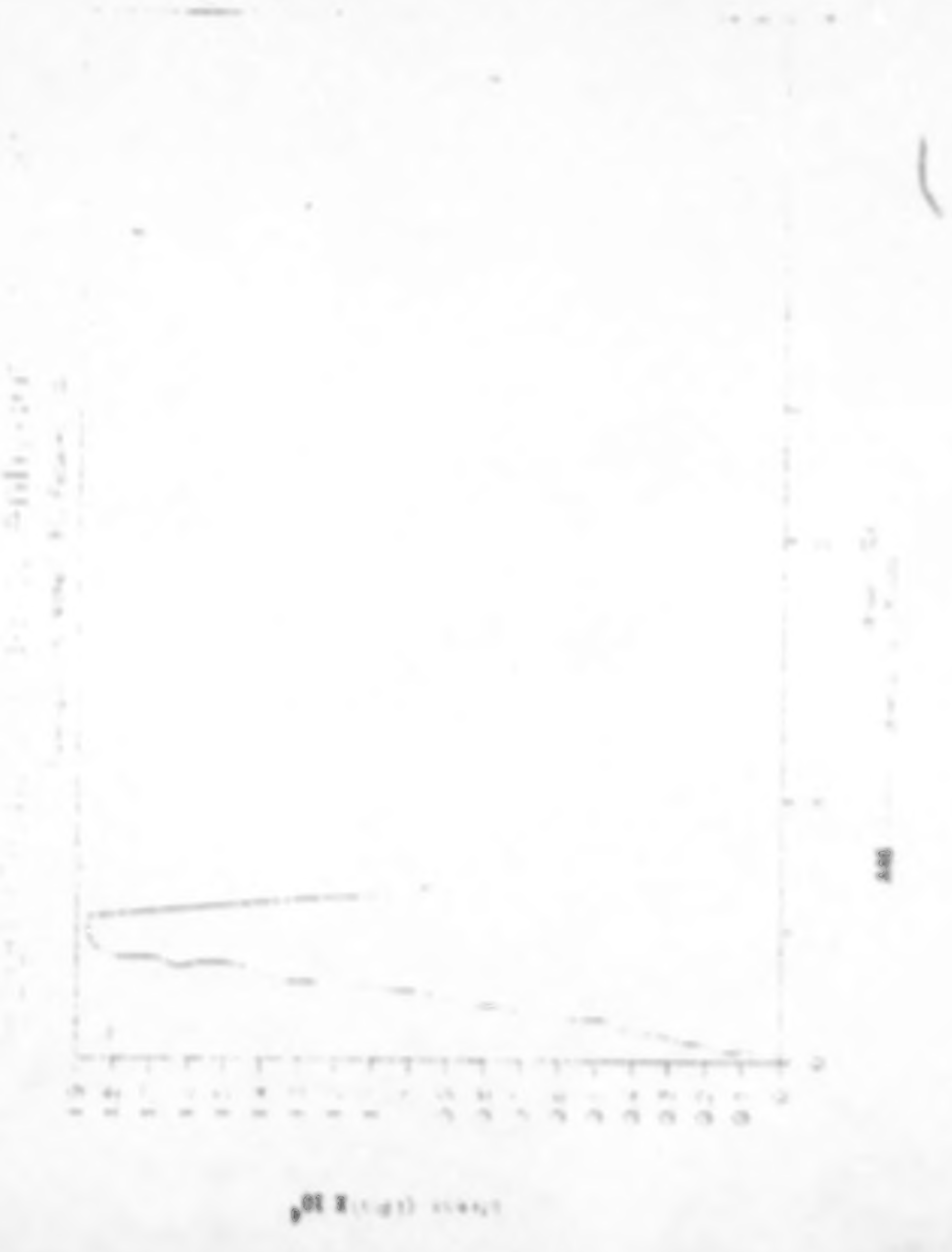
441
X 10^4

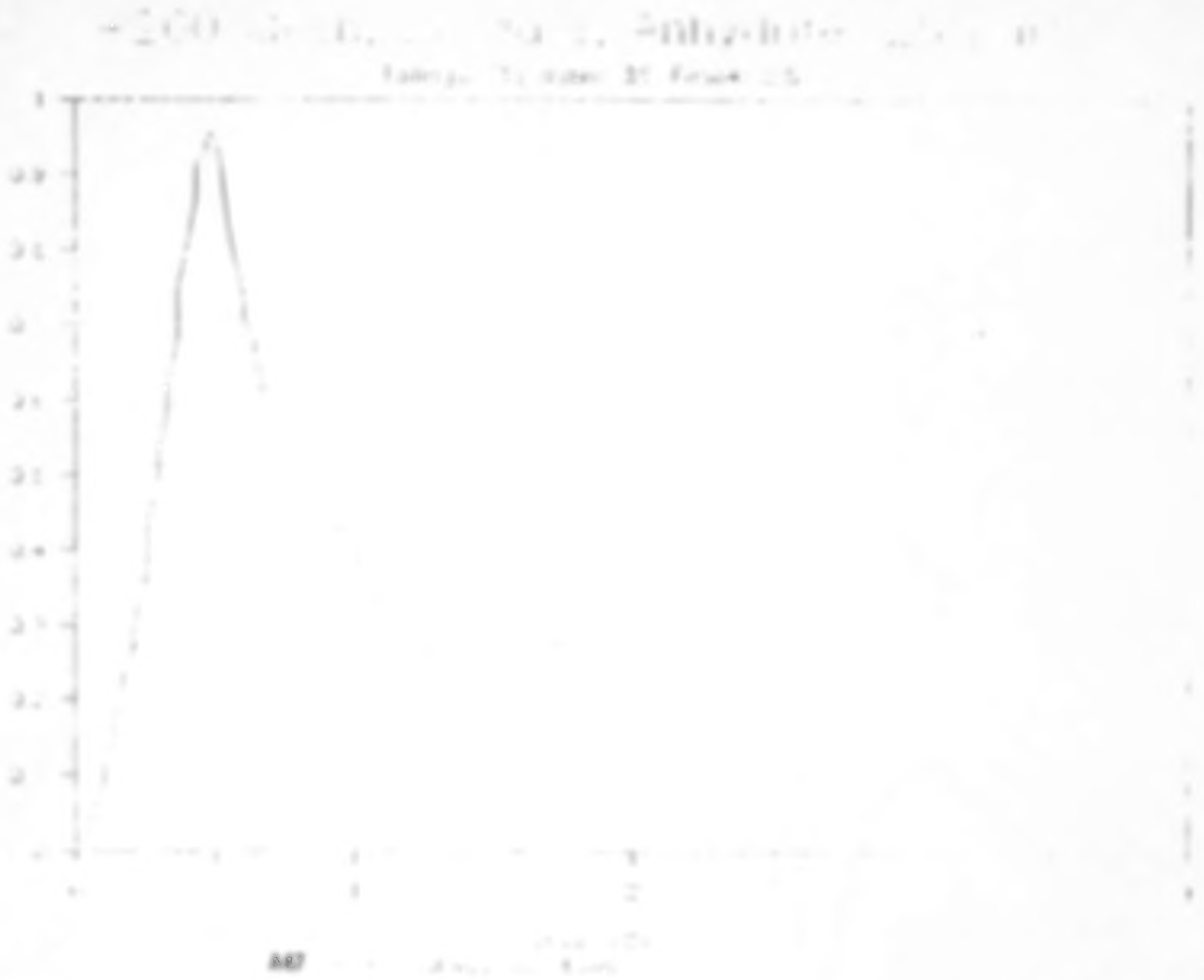
A42
x 10^4

A43
X 10^4

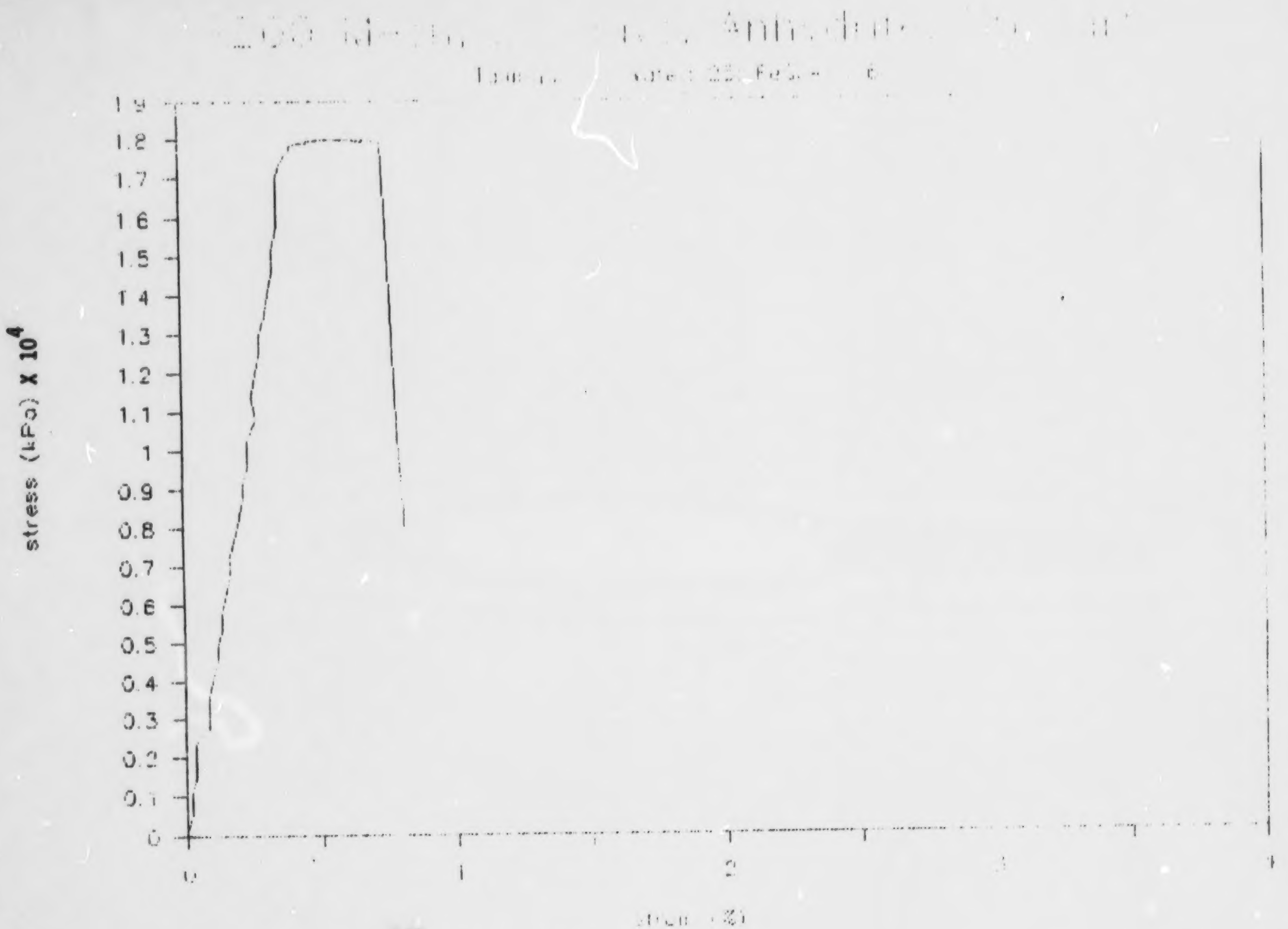
Stress (psi) X 10^3

A45

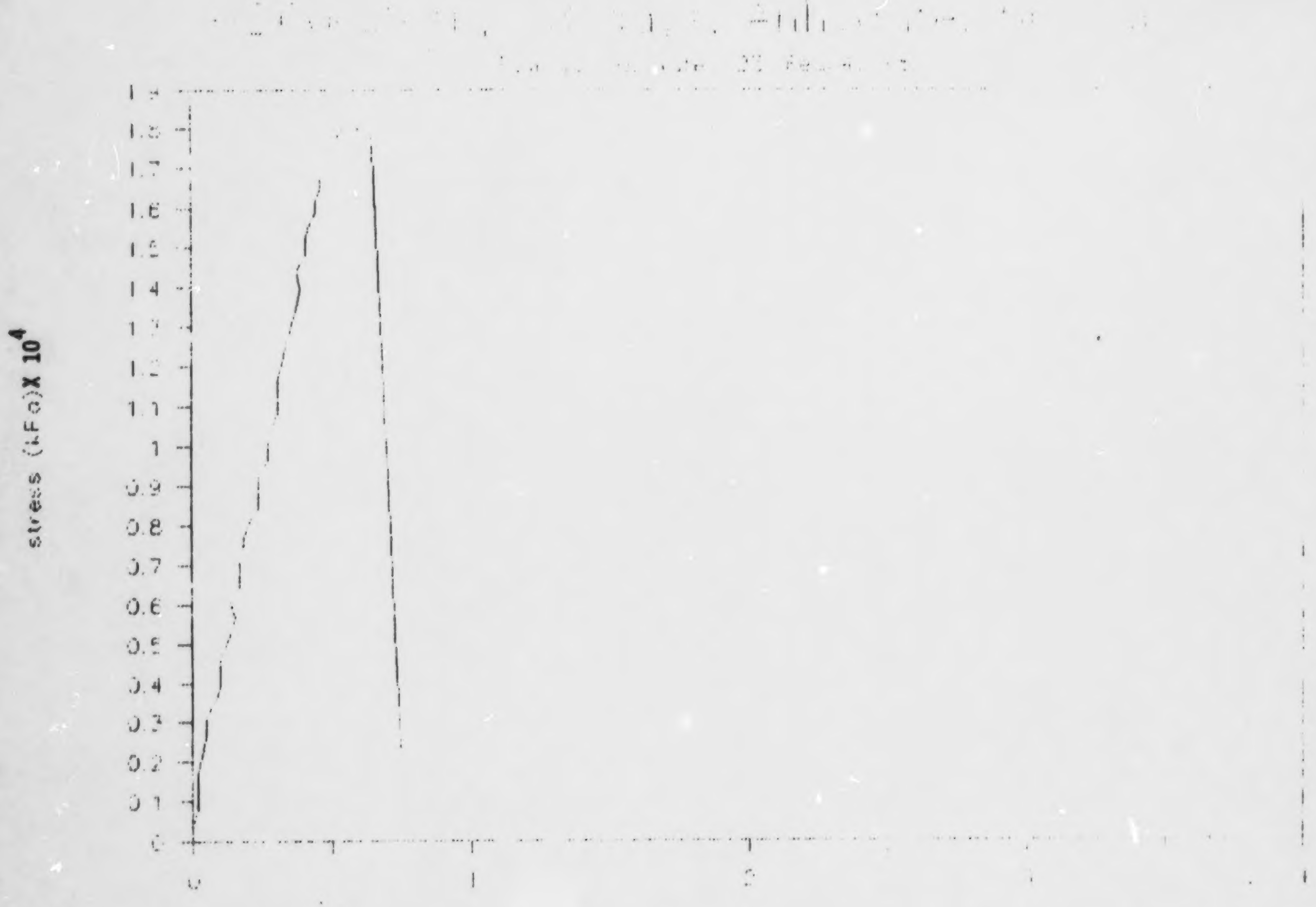




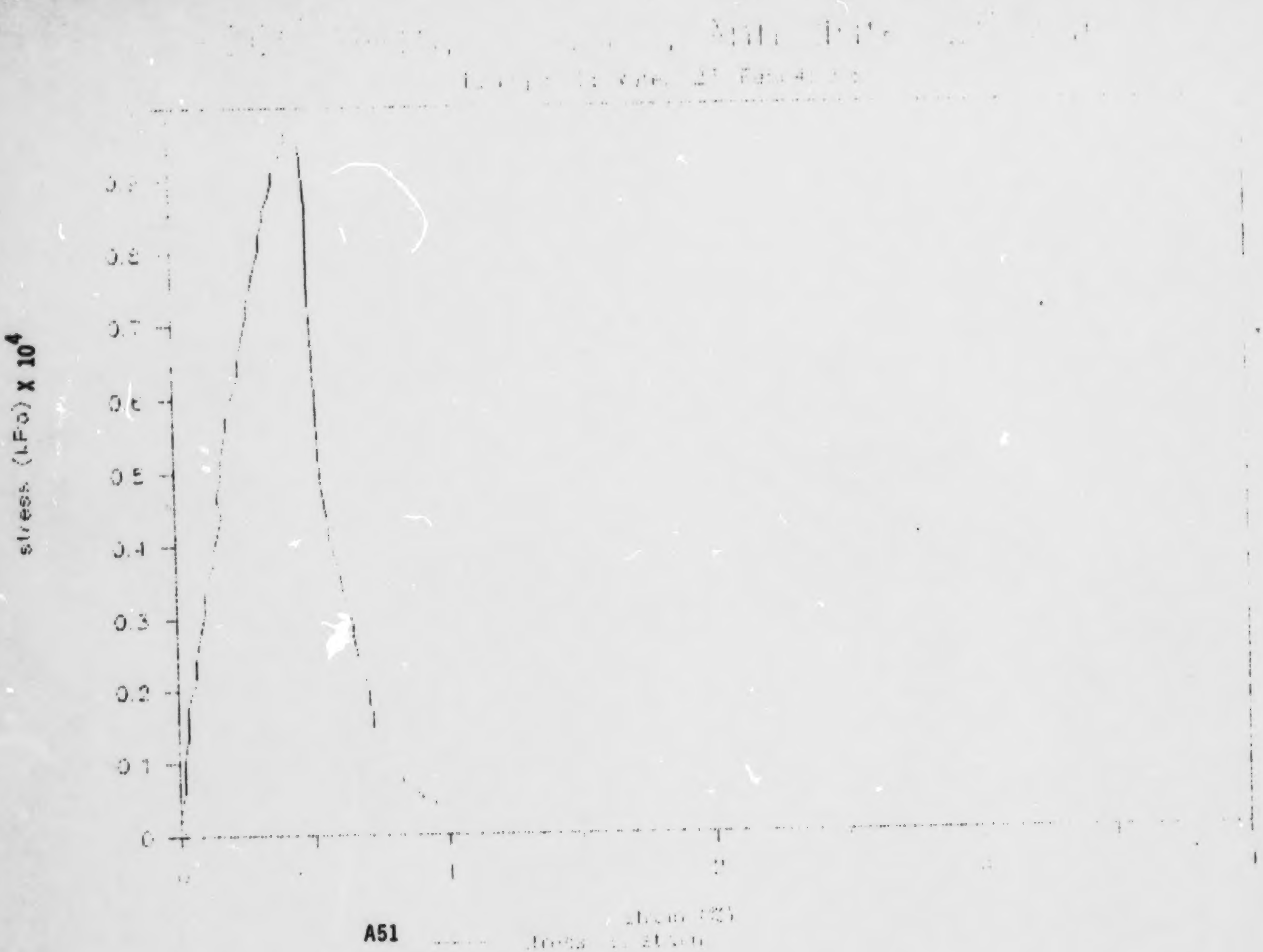
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
0
1
2


Anhydrite
stress (kPa) X 10^4
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4

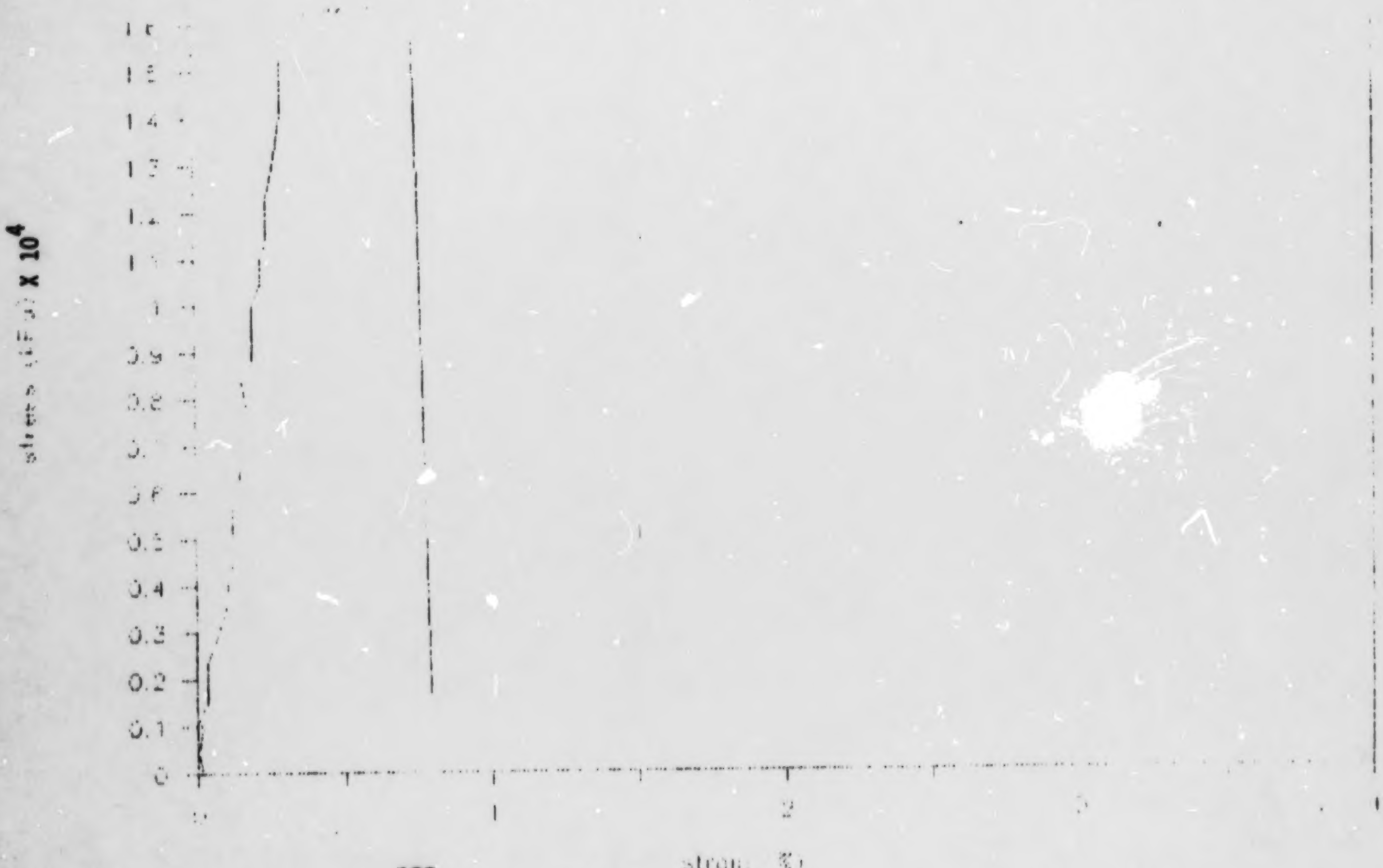
A49


stress (kPa)X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
0
1
2
3
4

A50

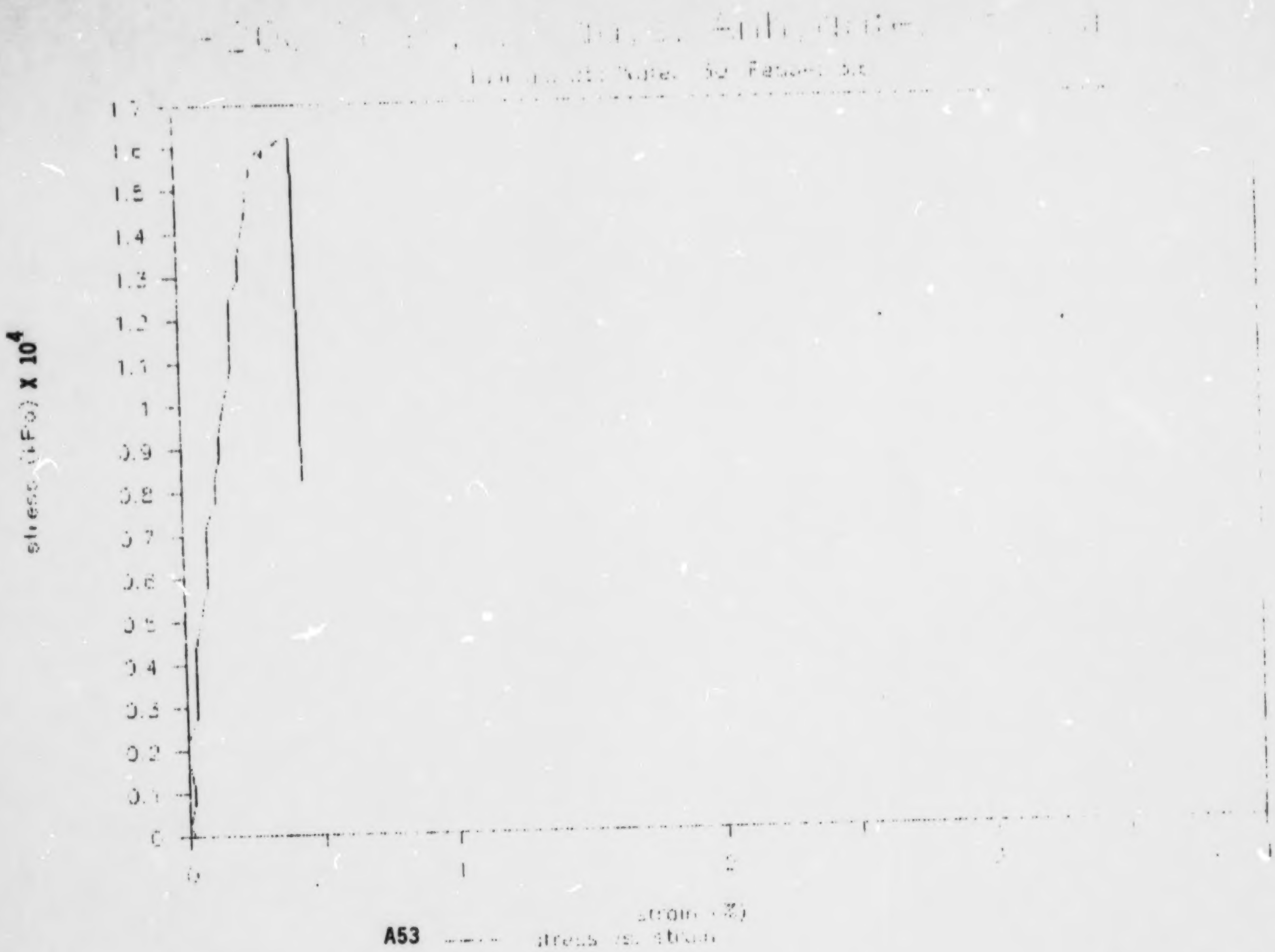


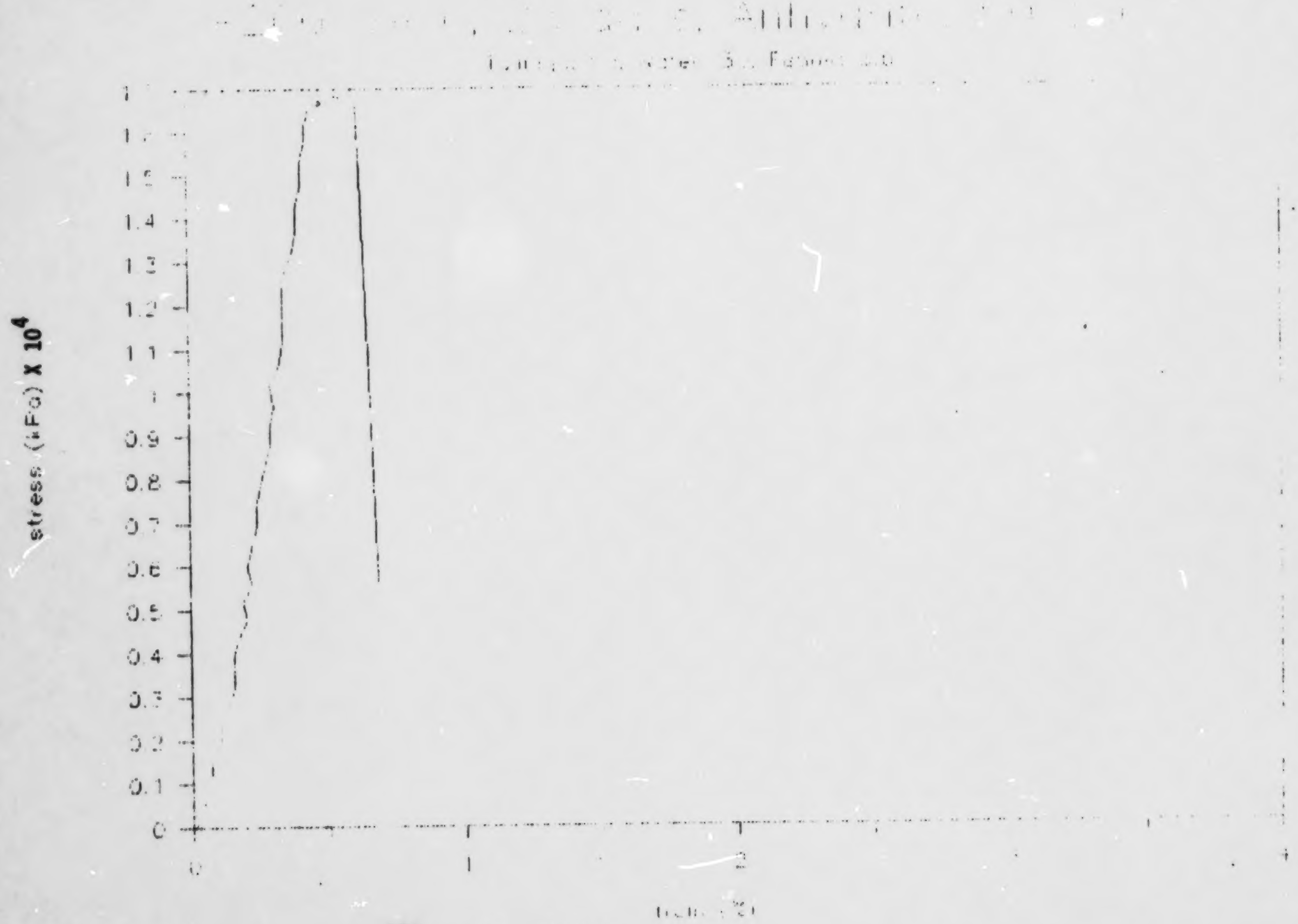
A51



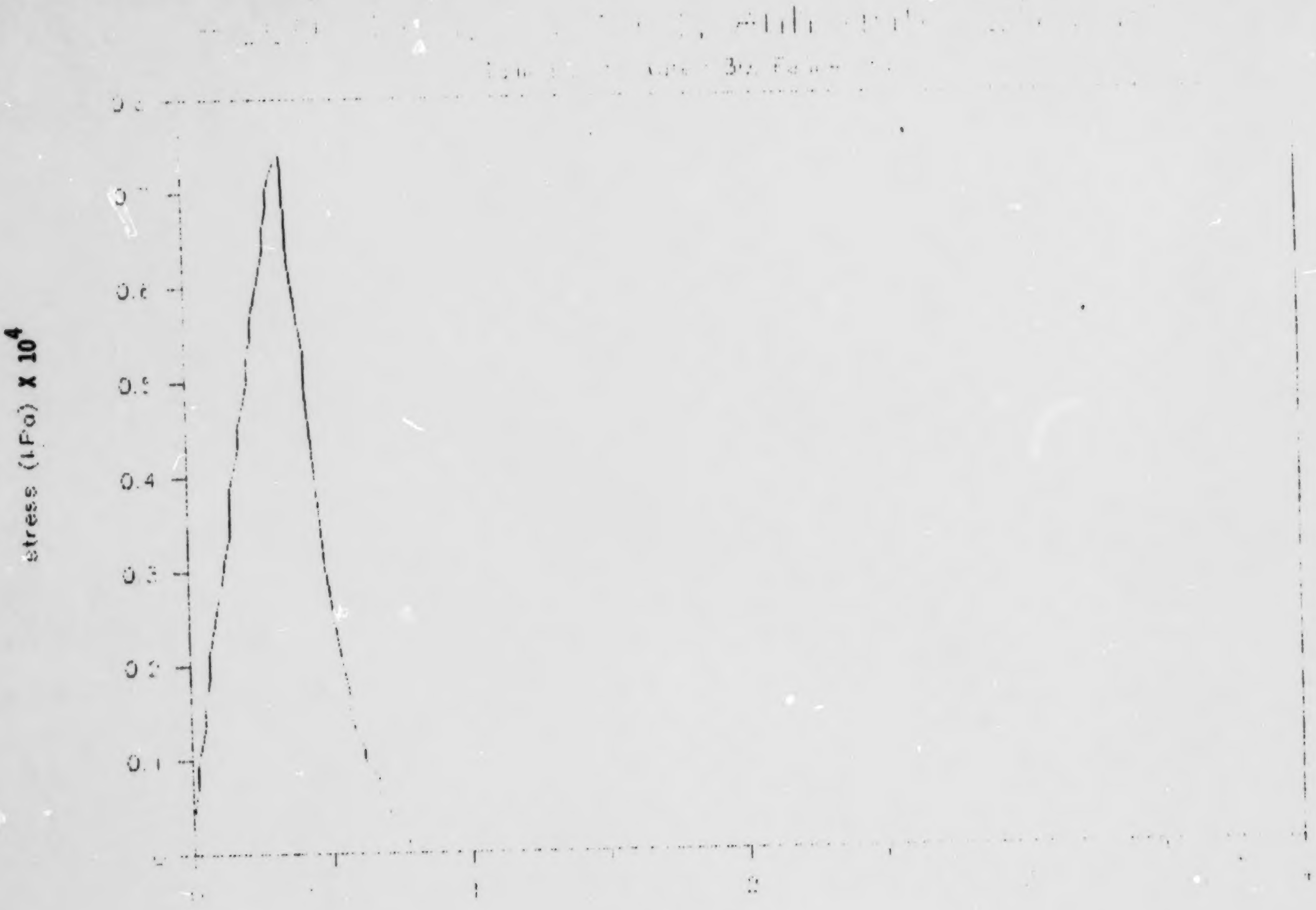
A52

stress – strain

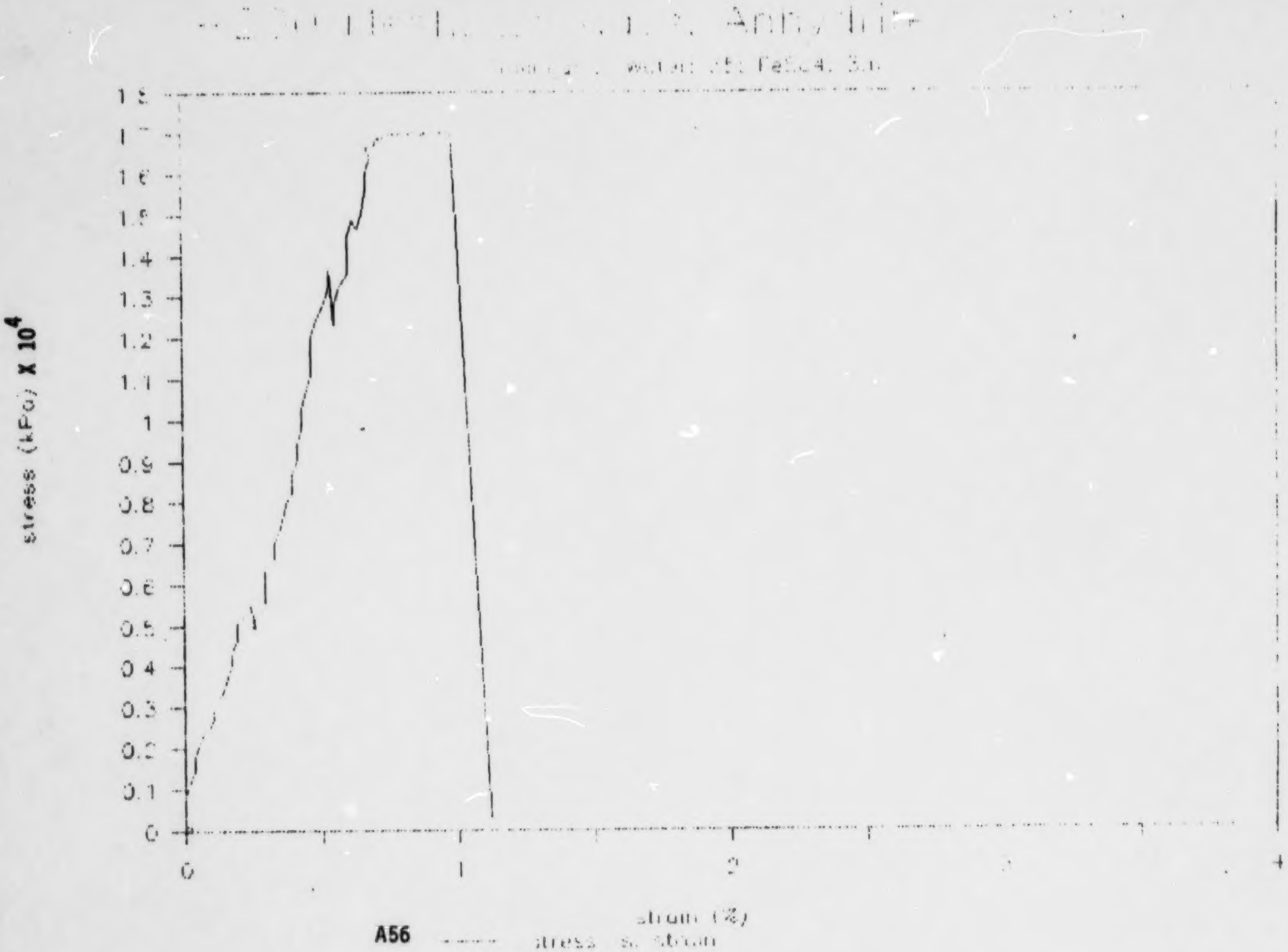

stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A53 stress vs. strain


stress (kPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
0
1
2
4
stress

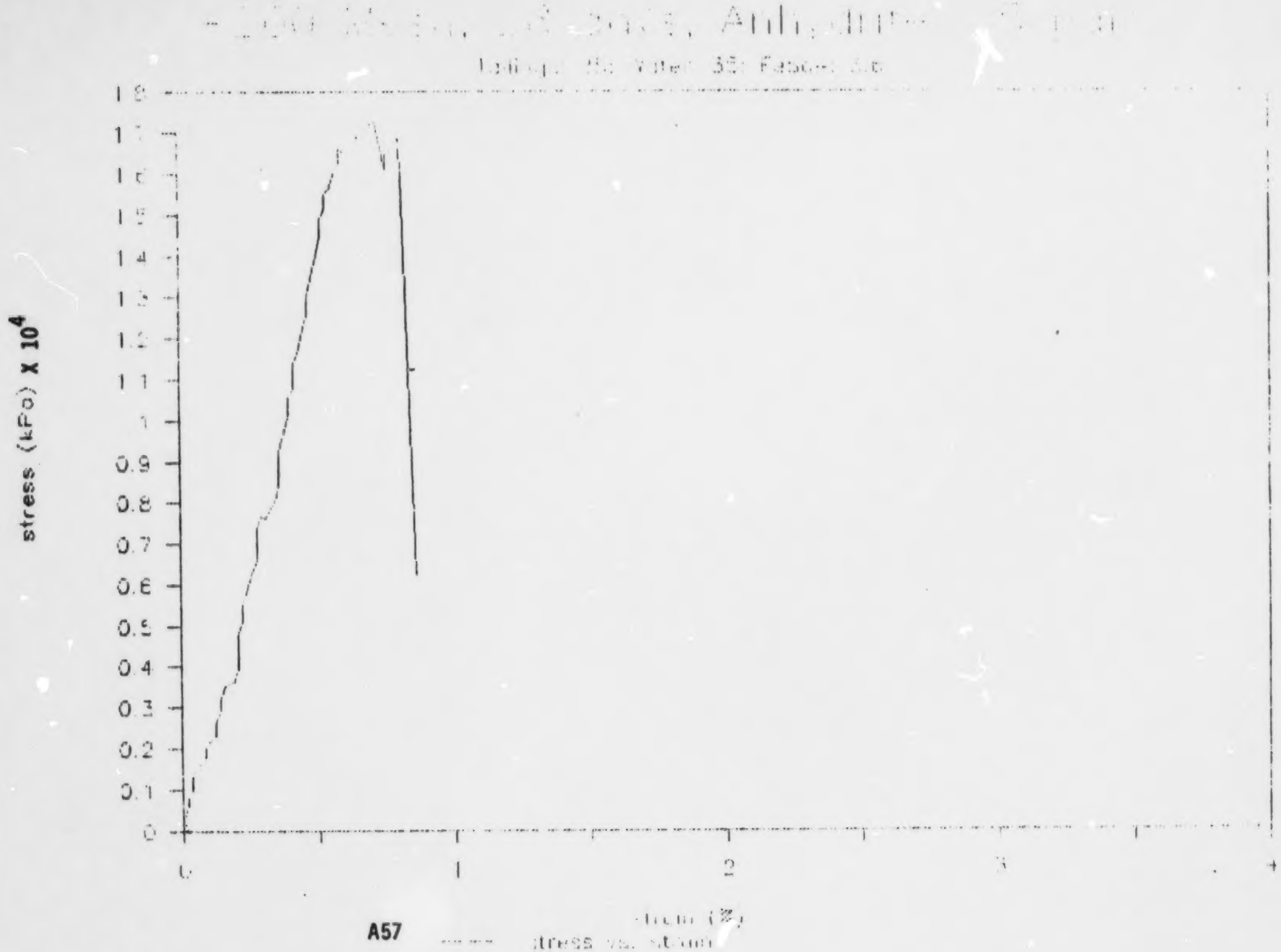
A54



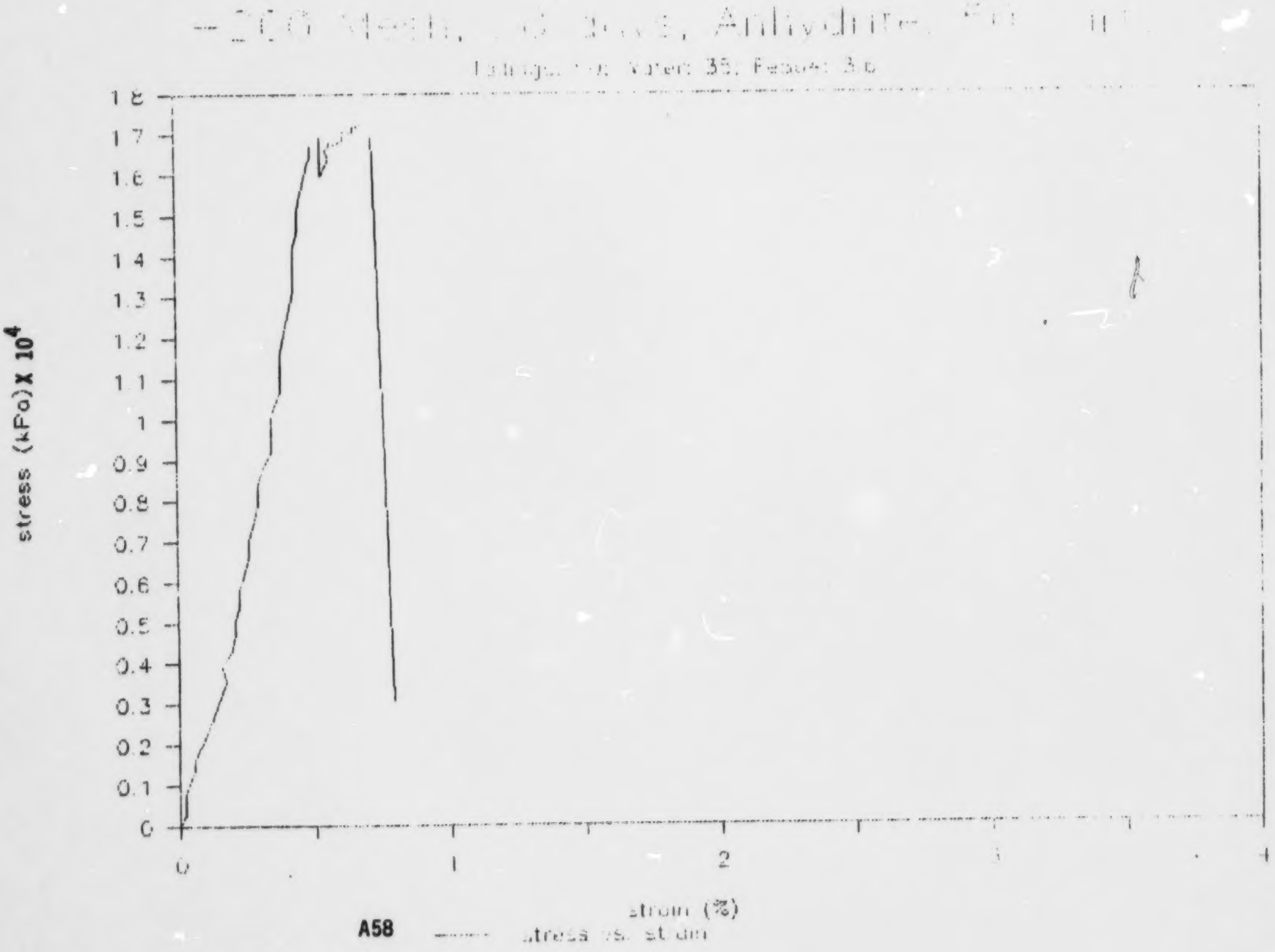
A55


stress (kPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
0
1
2
3
4
strain (%)
stress vs. strain

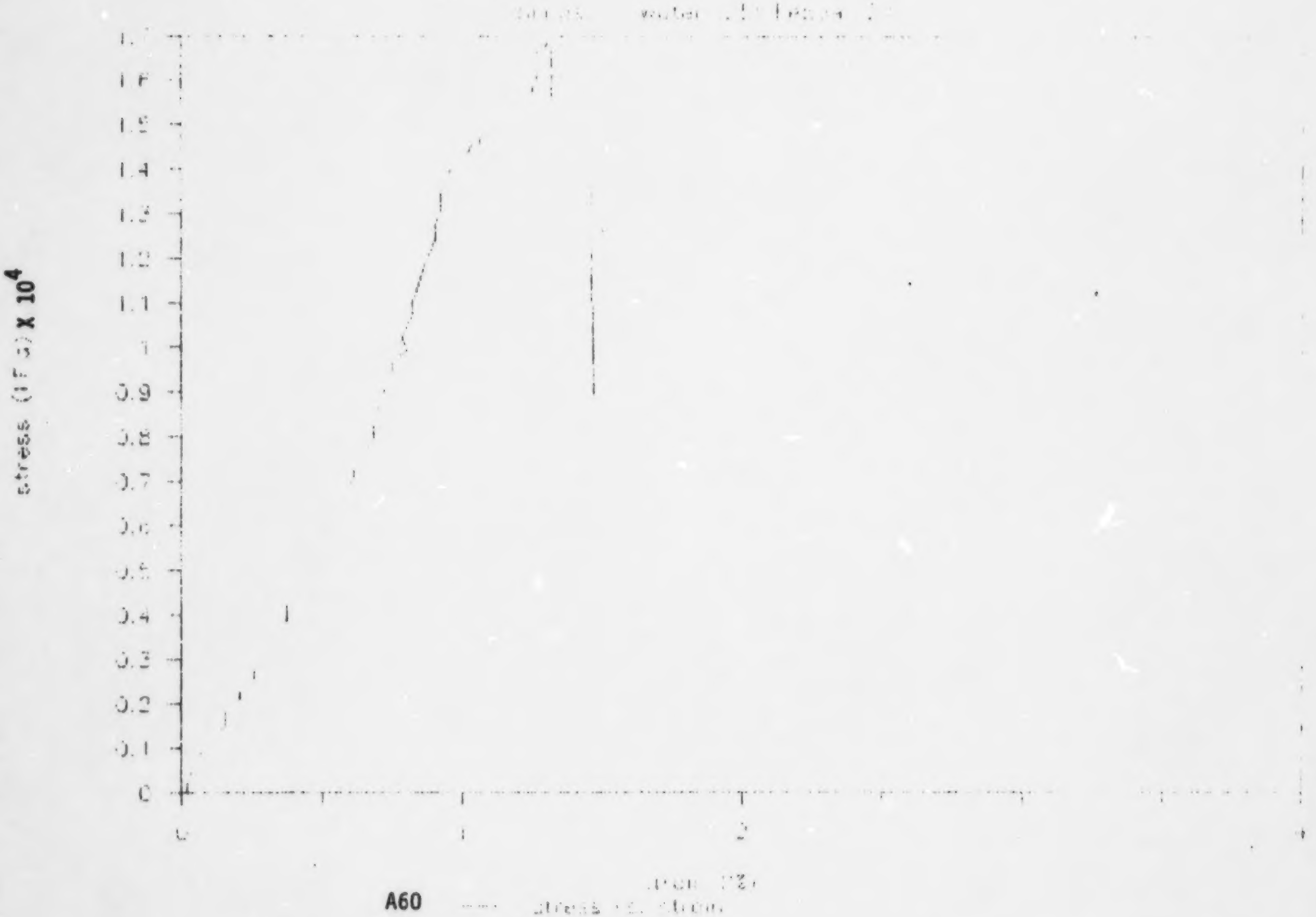
A56

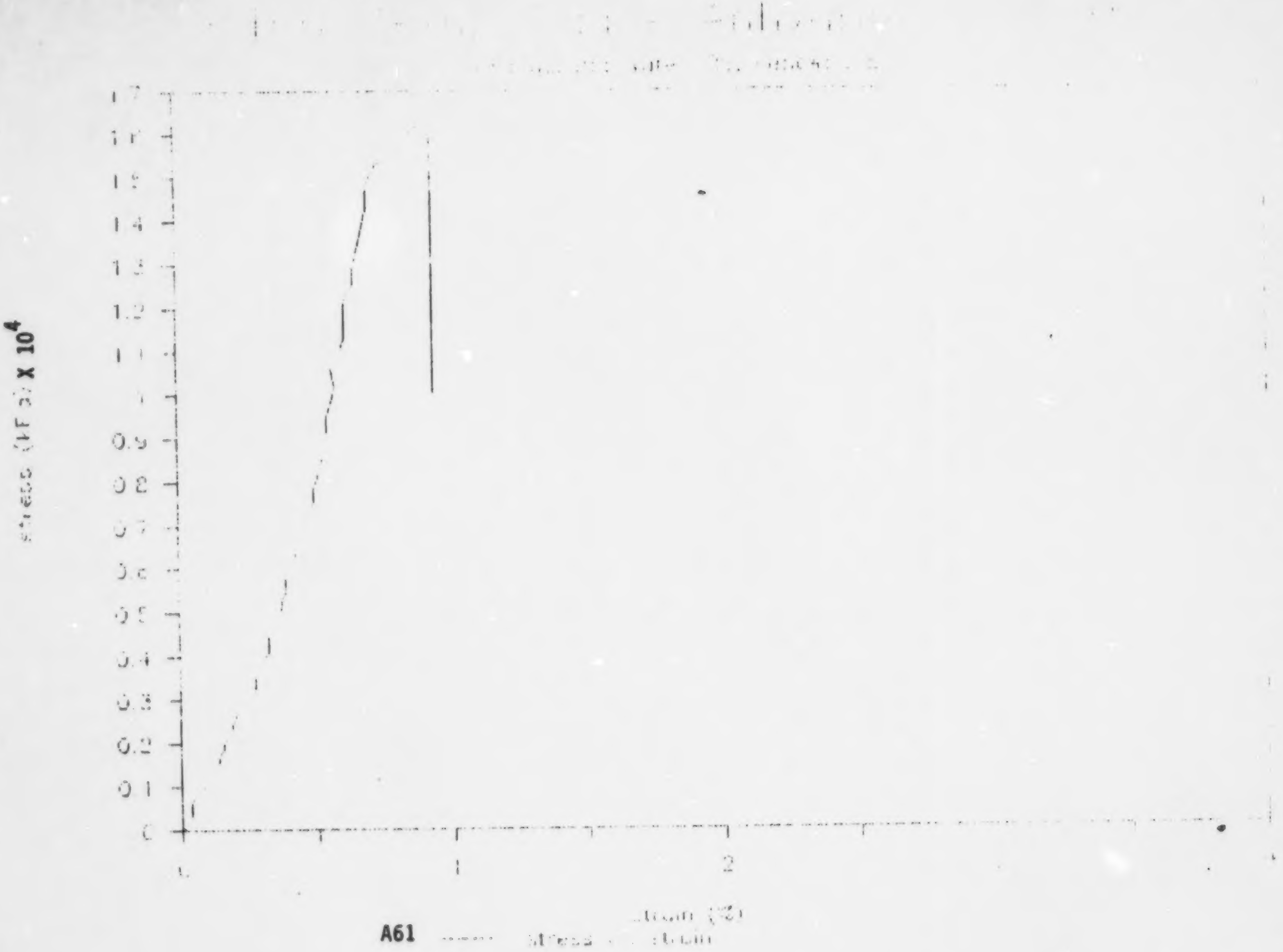

stress (kPa) X 10^4
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
stress vs. strain

A57

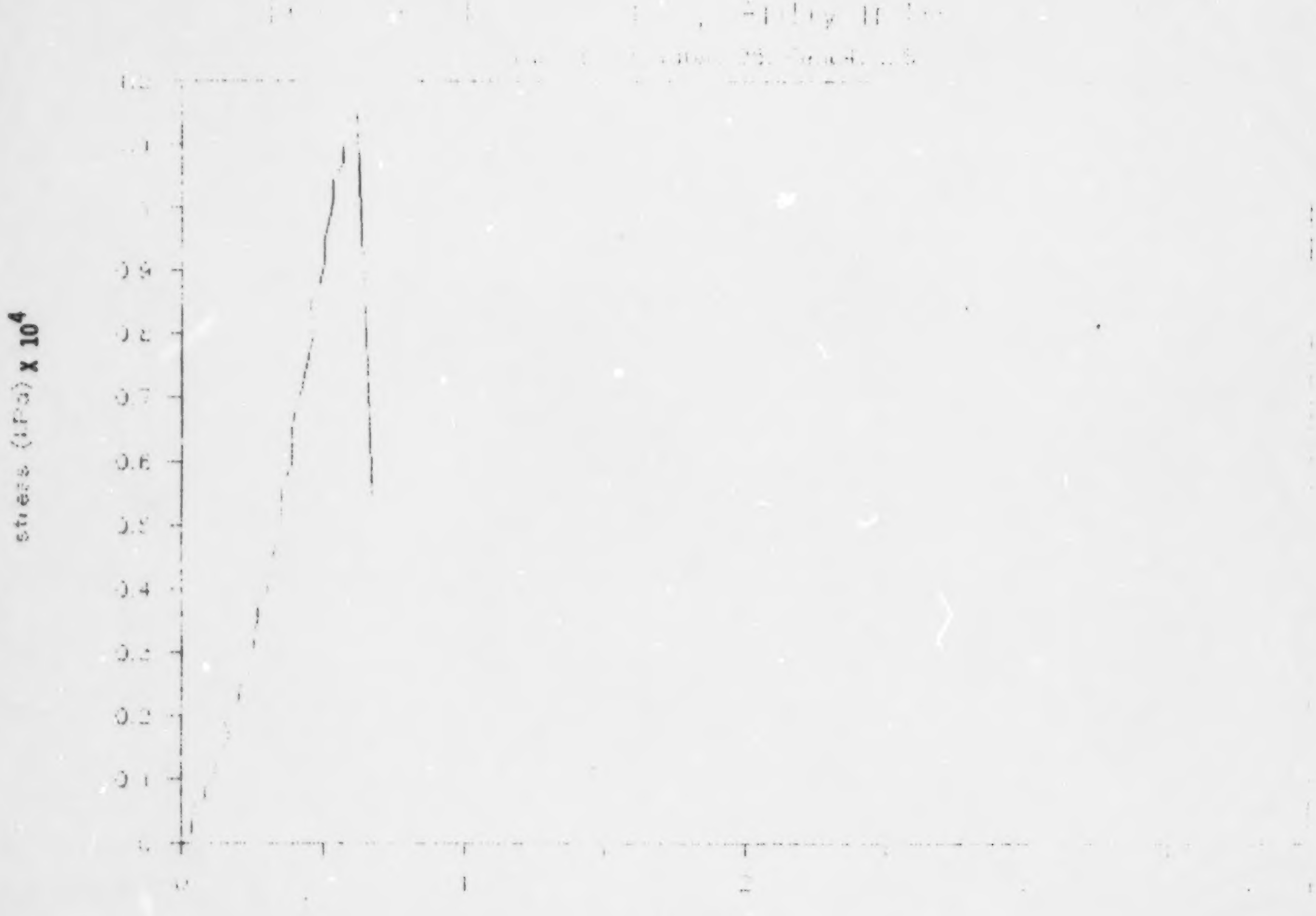

-200 Mesh, [illegible] Days, Anhydrite [illegible]
[illegible] Water: 35; [illegible] 3.6
stress (kPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
0
1
2
3
4
strain (%)
A58
stress vs. strain

A59

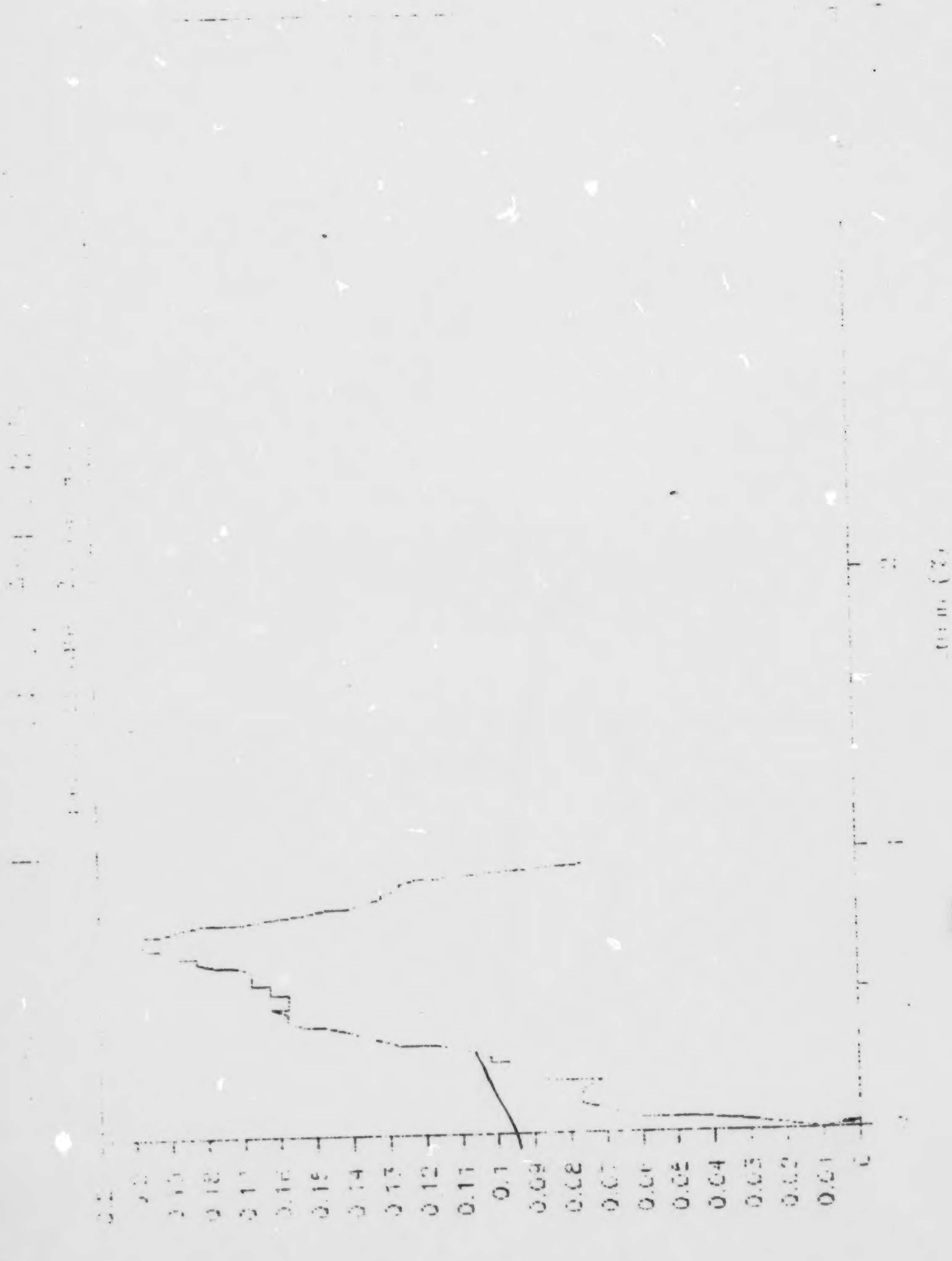

stress (1F 3) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
0
1
2
strain (%)
A60
stress vs. strain



Stress
X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
0
1
2
Strain (%)
A61
Stress - Strain

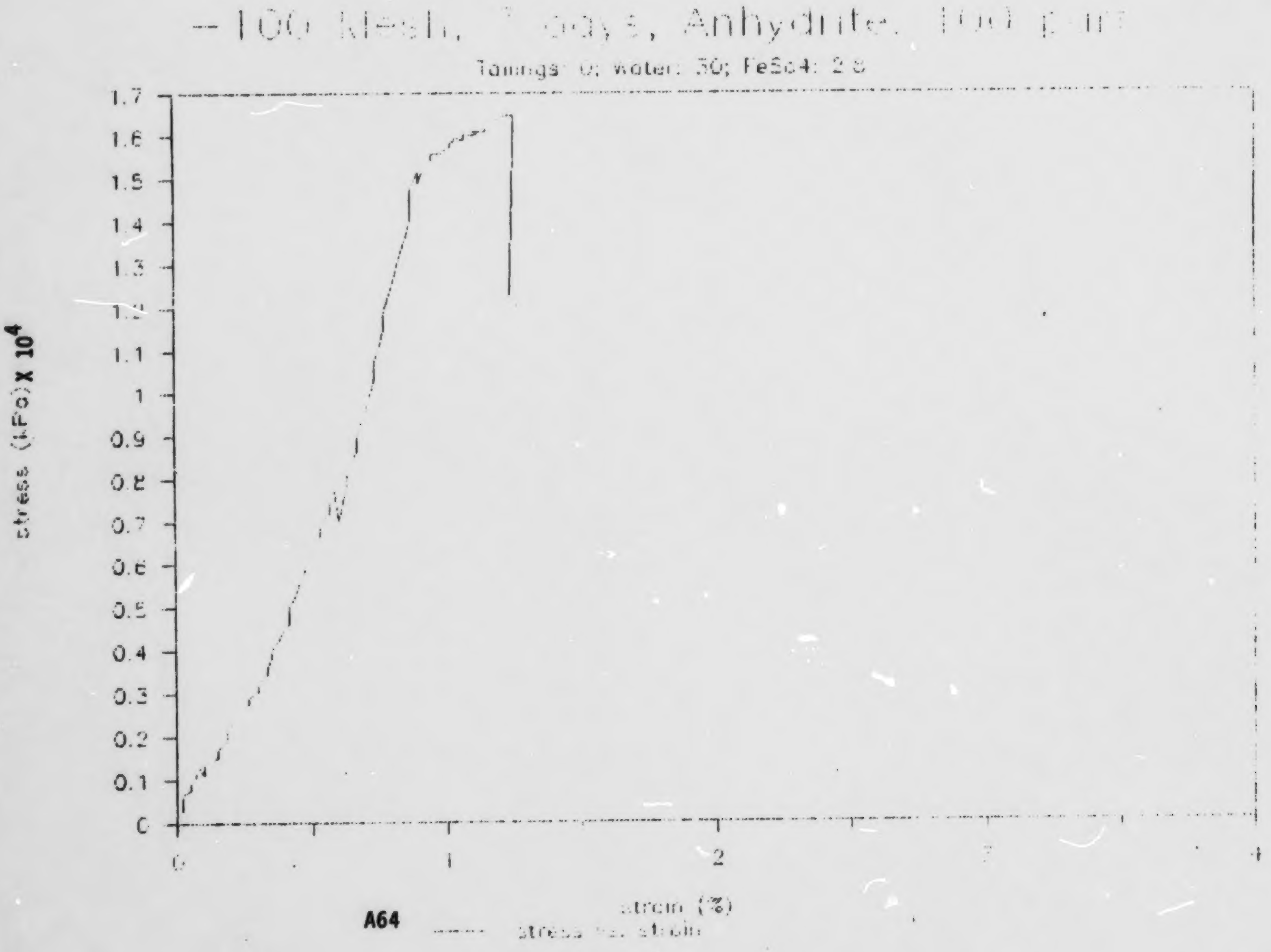


A62

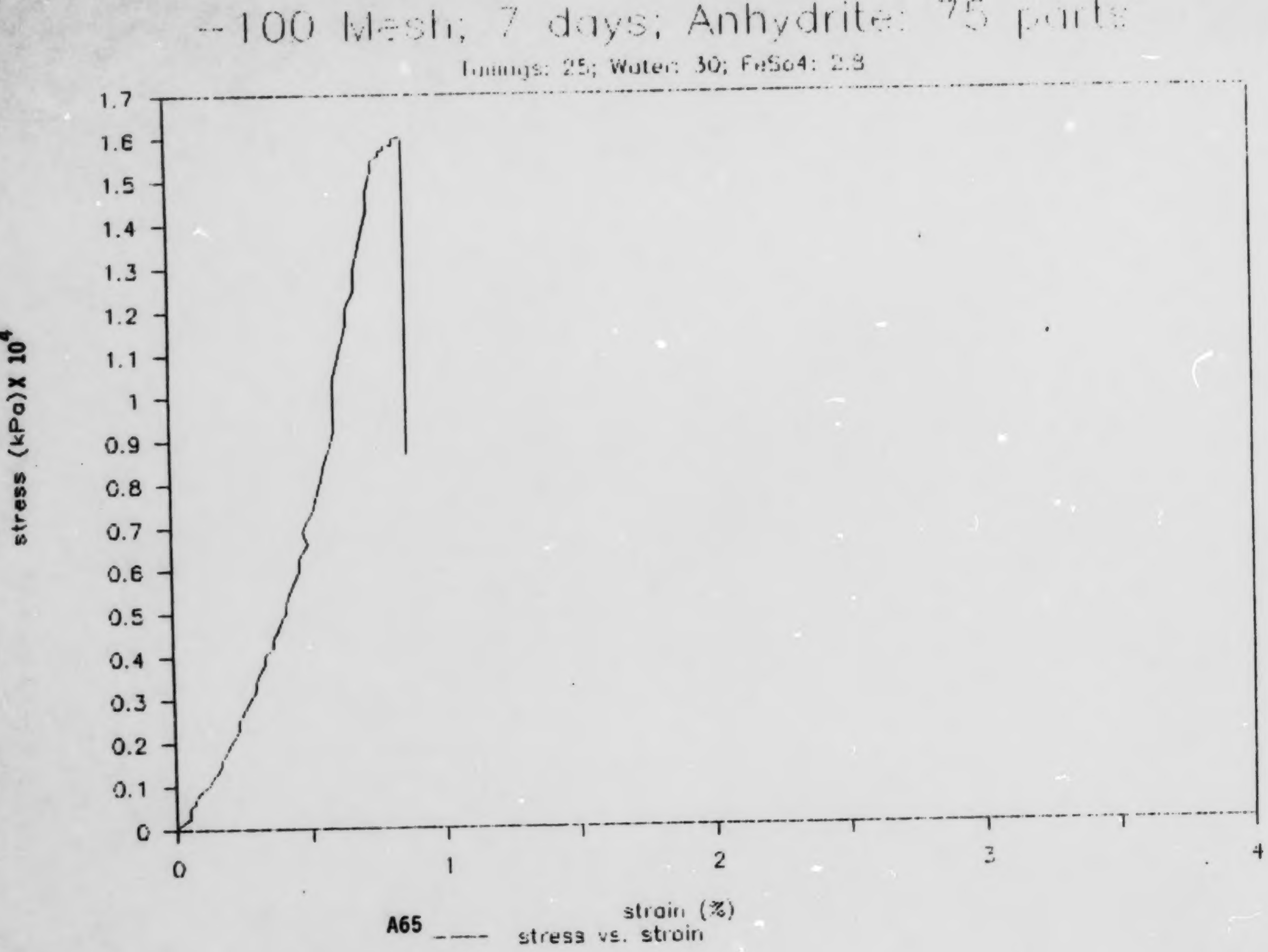


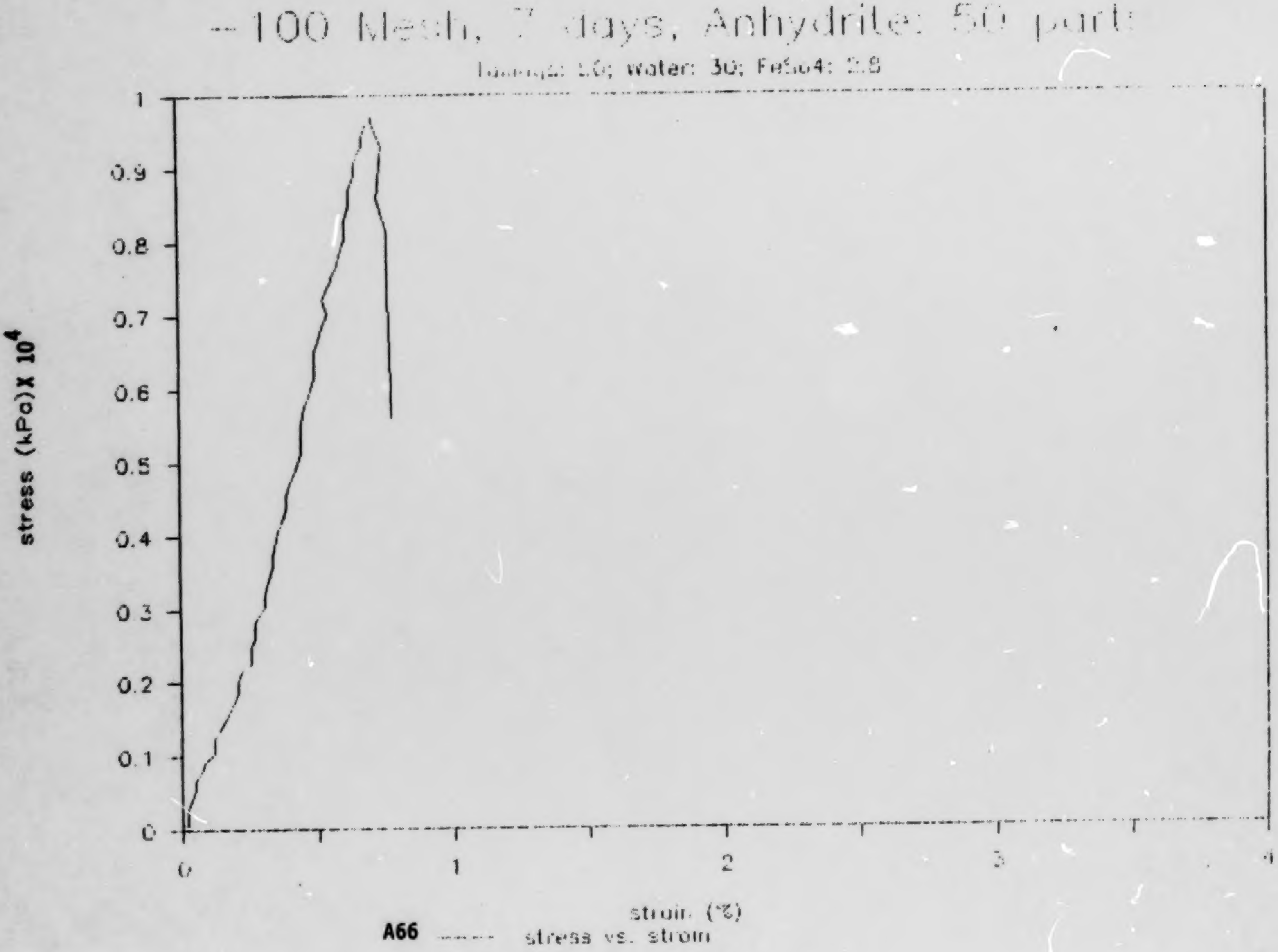
stress (kPa) X 10^4
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.1
0.11
0.12
0.13
0.14
0.15
0.16
0.17
0.18
0.19
0.2

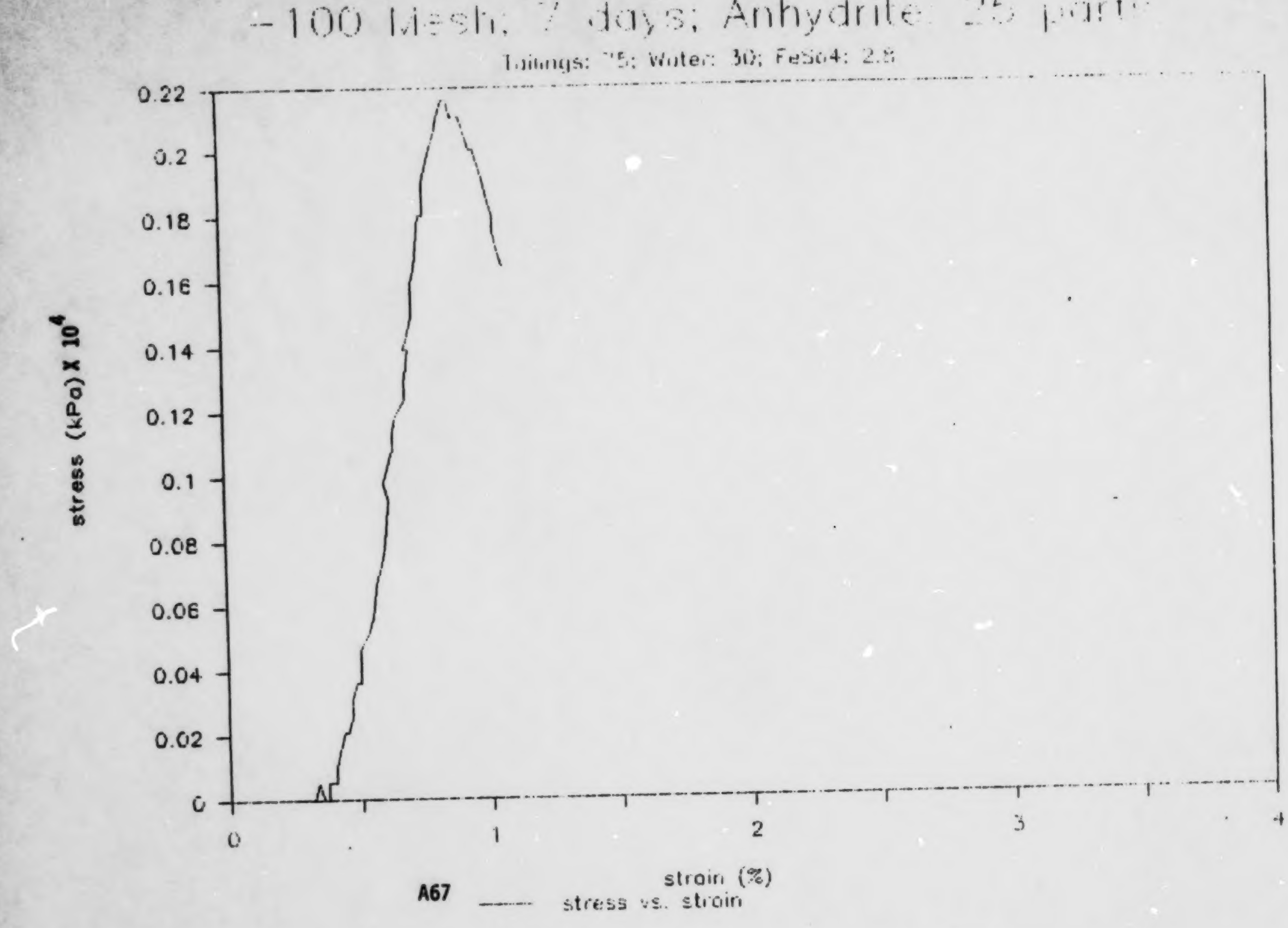
A63


−100 Mesh, 7 days, Anhydrite, 100 psi
Tailings: 0; water: 30; FeSo4: 2.8
stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
stress vs. strain

A64

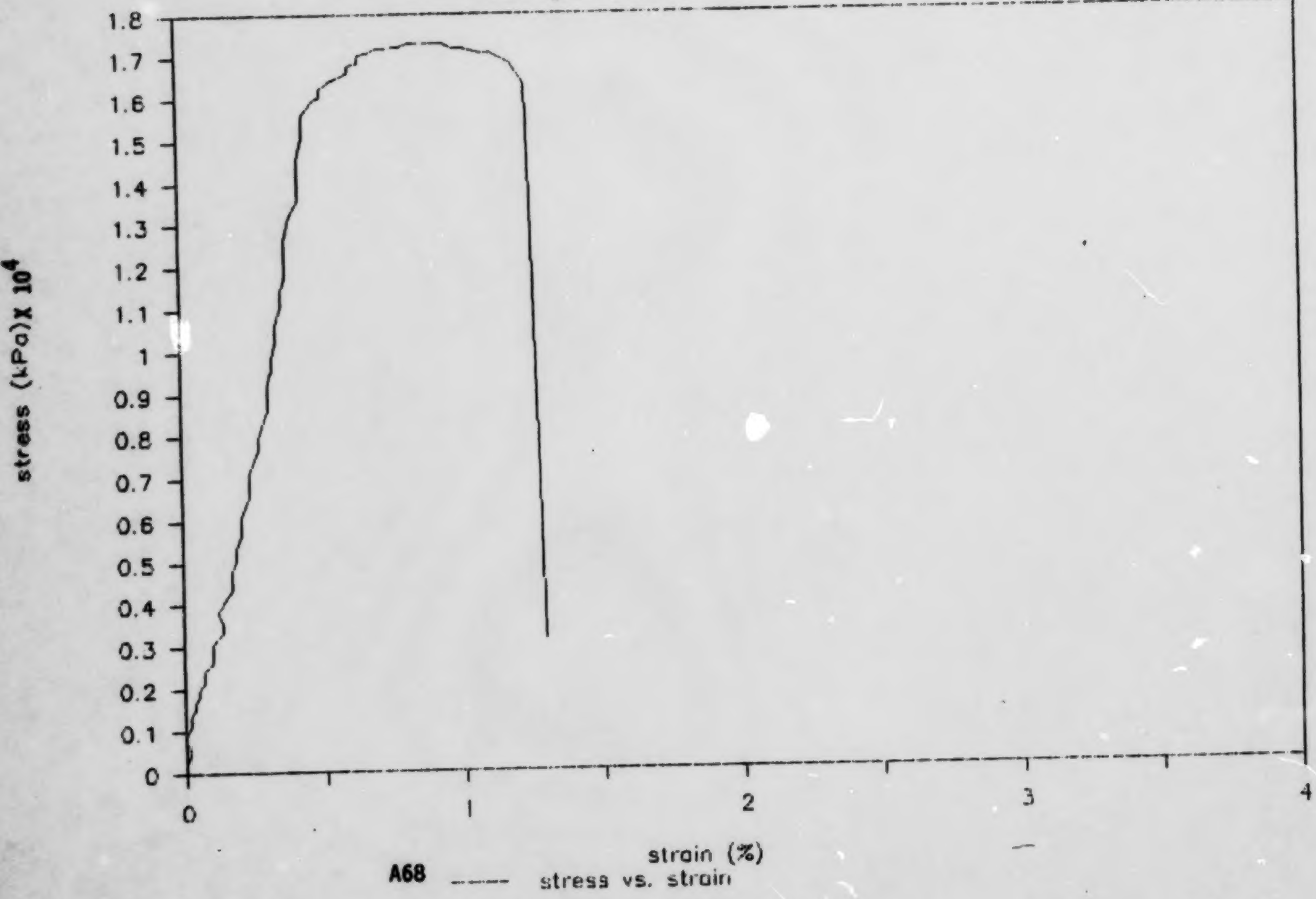

-100 Mesh; 7 days; Anhydrite: 75 parts
Tailings: 25; Water: 30; FeSo4: 2.8
stress (kPa) X 10⁴
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A65
stress vs. strain


-100 Mesh, 7 days, Anhydrite: 50 part
Tailings: 50; water: 30; FeSO4: 2.8
stress (kPa) X 10^4
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A66 ----- stress vs. strain


-100 Mesh; 7 days; Anhydrite: 25 parts
Tailings: 75; Water: 30; FeSo4: 2.8
stress (kPa) X 10^4
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
3
4
strain (%)
A67
stress vs. strain

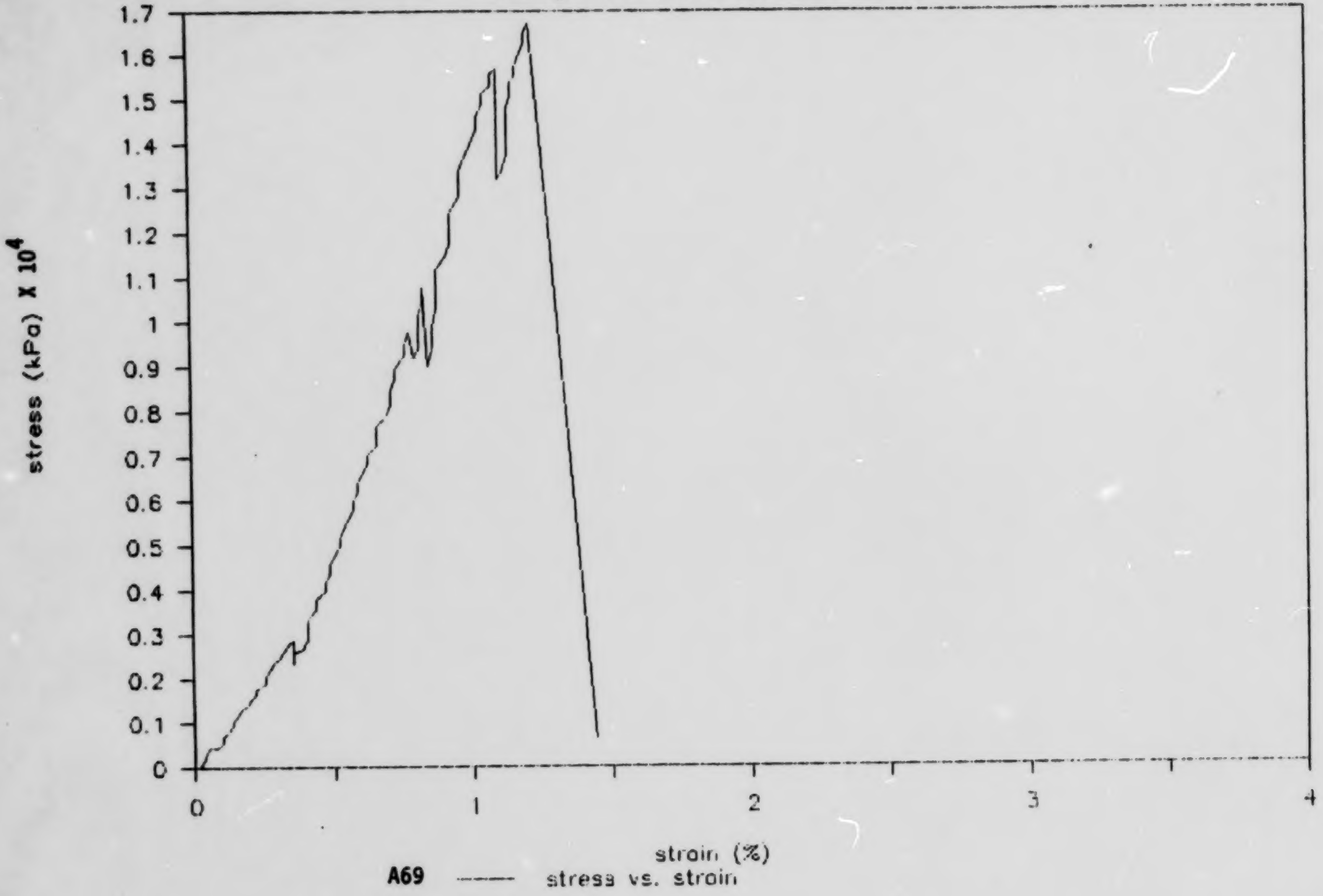
--100 Mesh; 7 days; Anhydrite: 100 parts

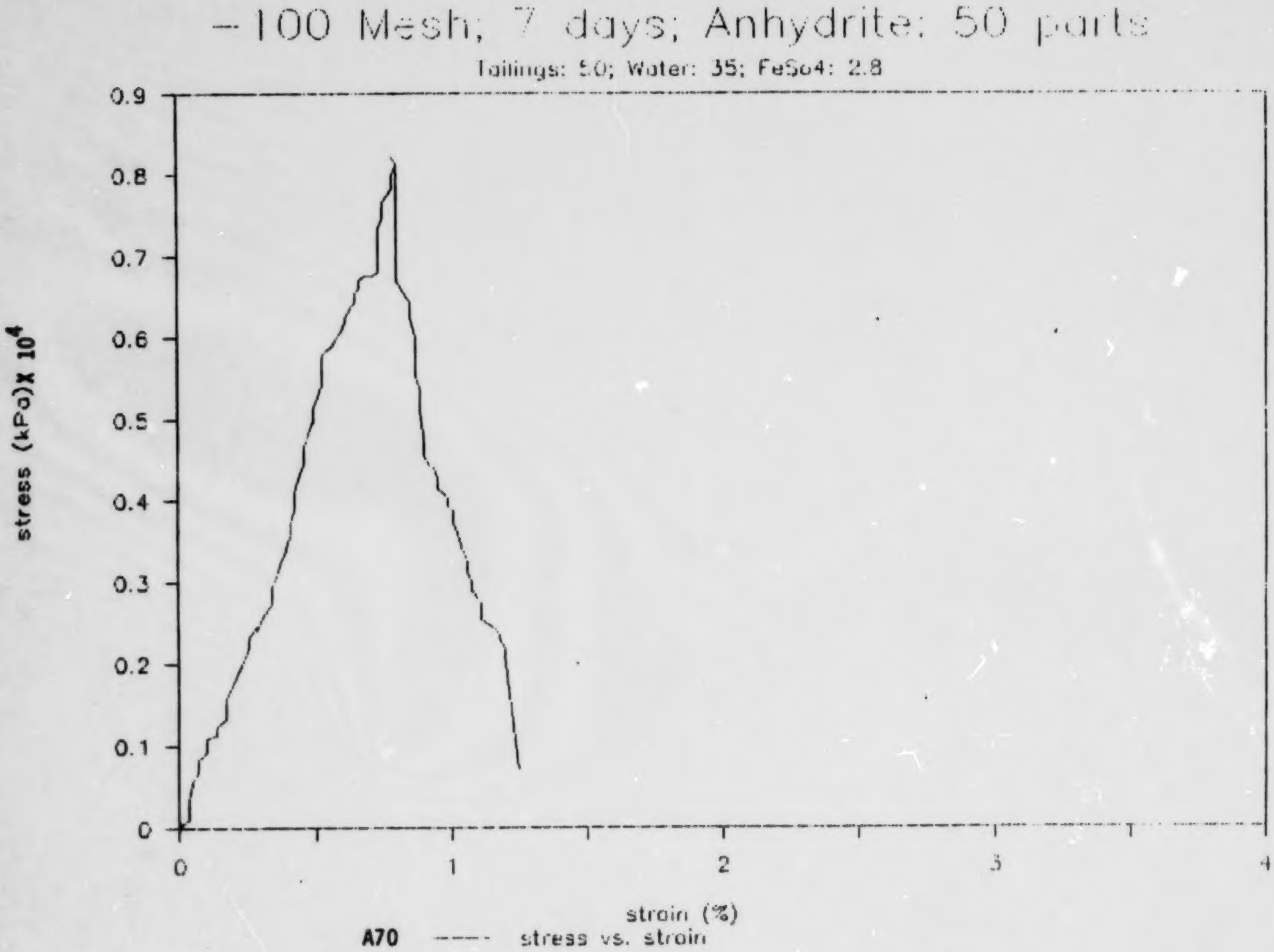
Tailings: 0; Water: 35; FeSo4: 2.8



-100 Mesh; 7 days; Anhydrite: 75 parts

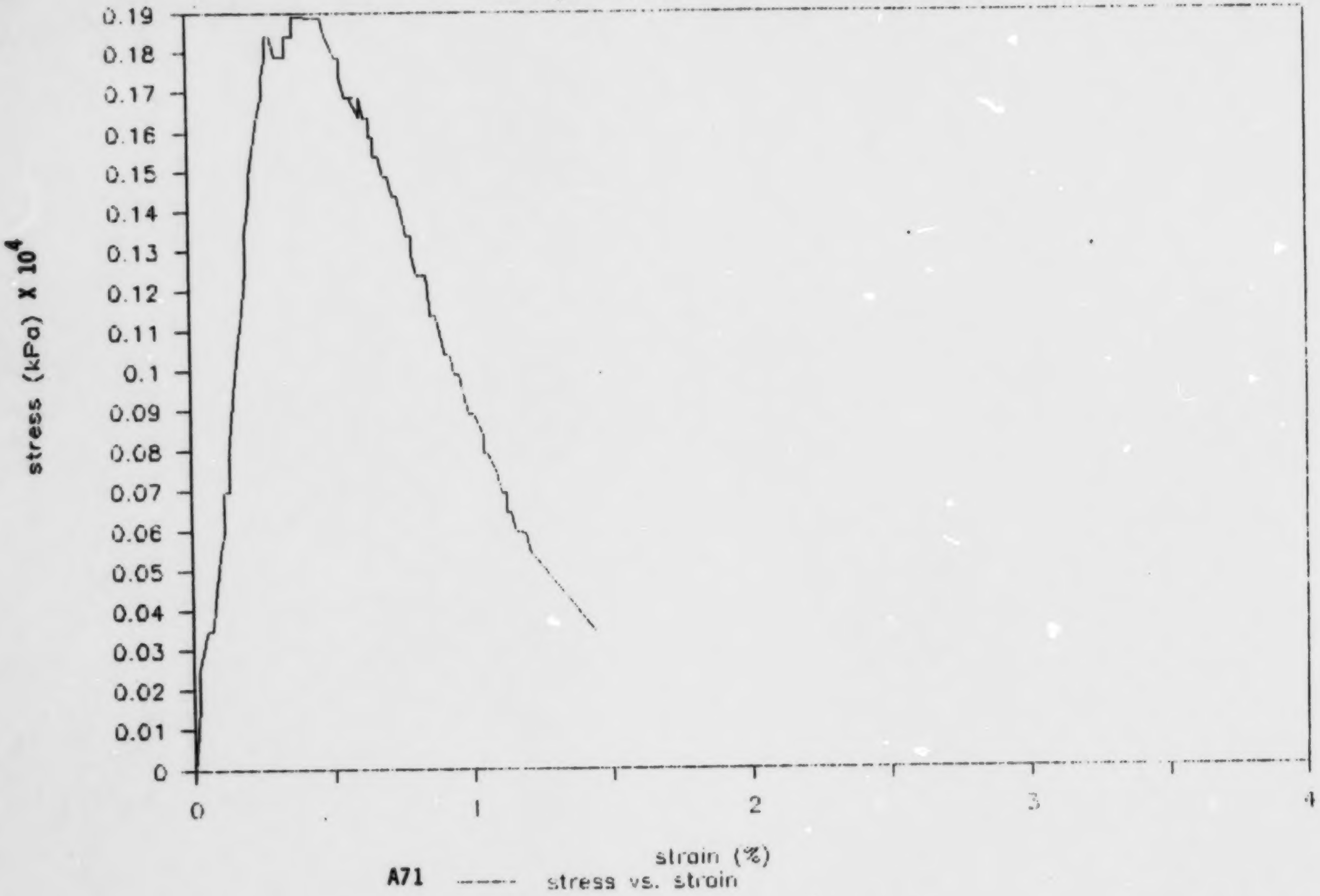
Tailings: 25; Water: 35; FeSo4: 2.8

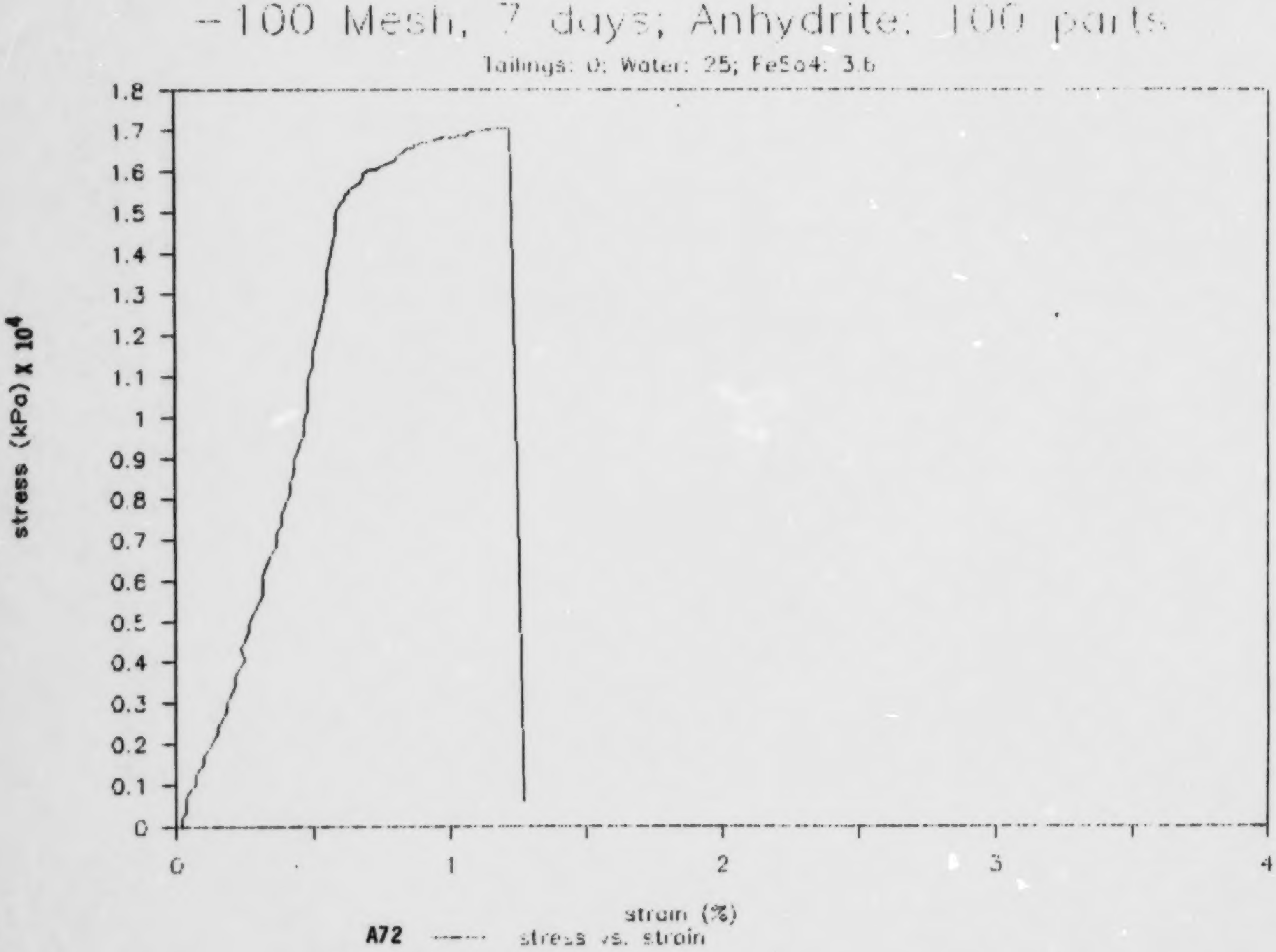


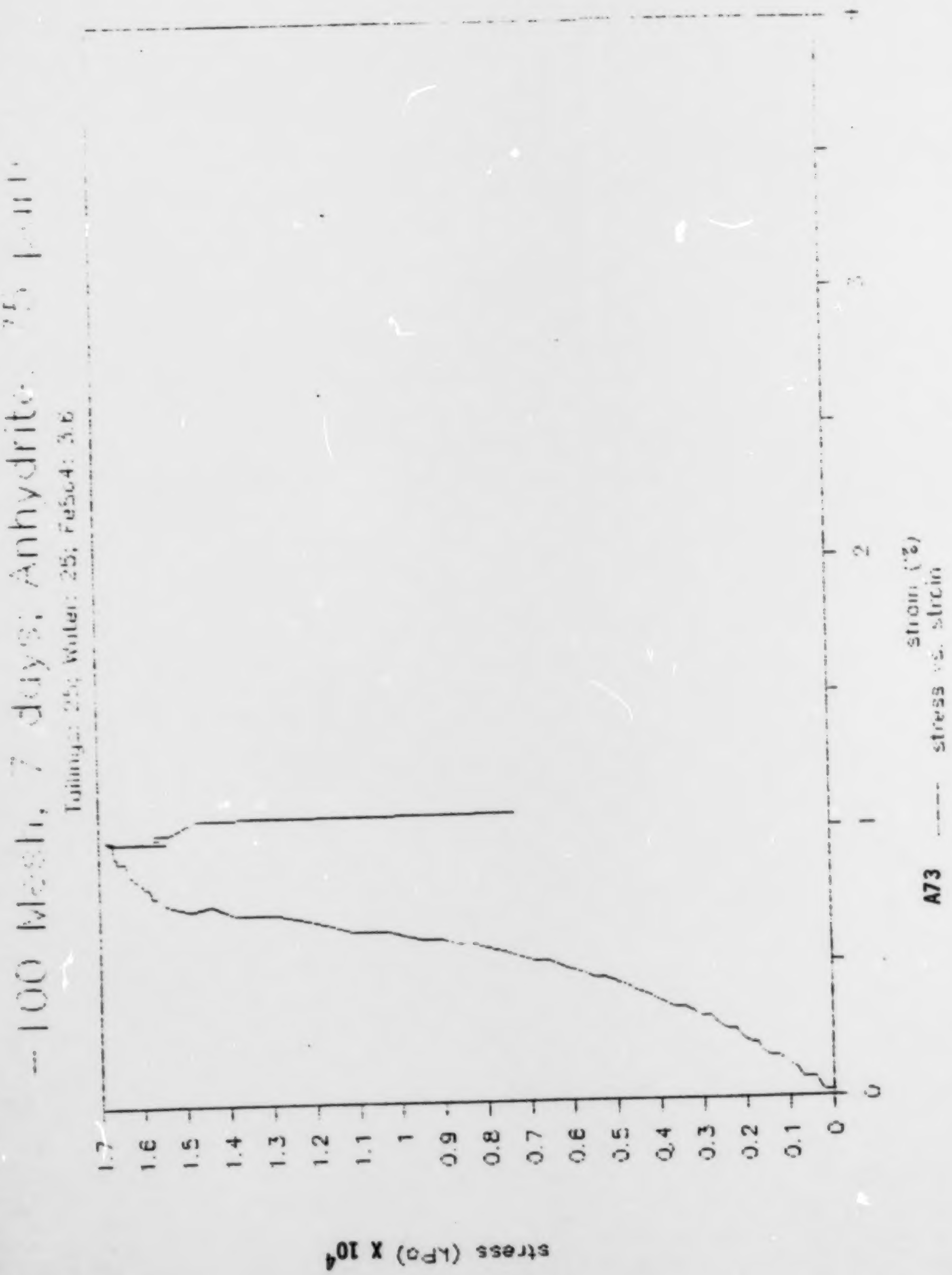

−100 Mesh; 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 35; FeSo4: 2.8
stress (kPa)X 10^4
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A70 ---- stress vs. strain

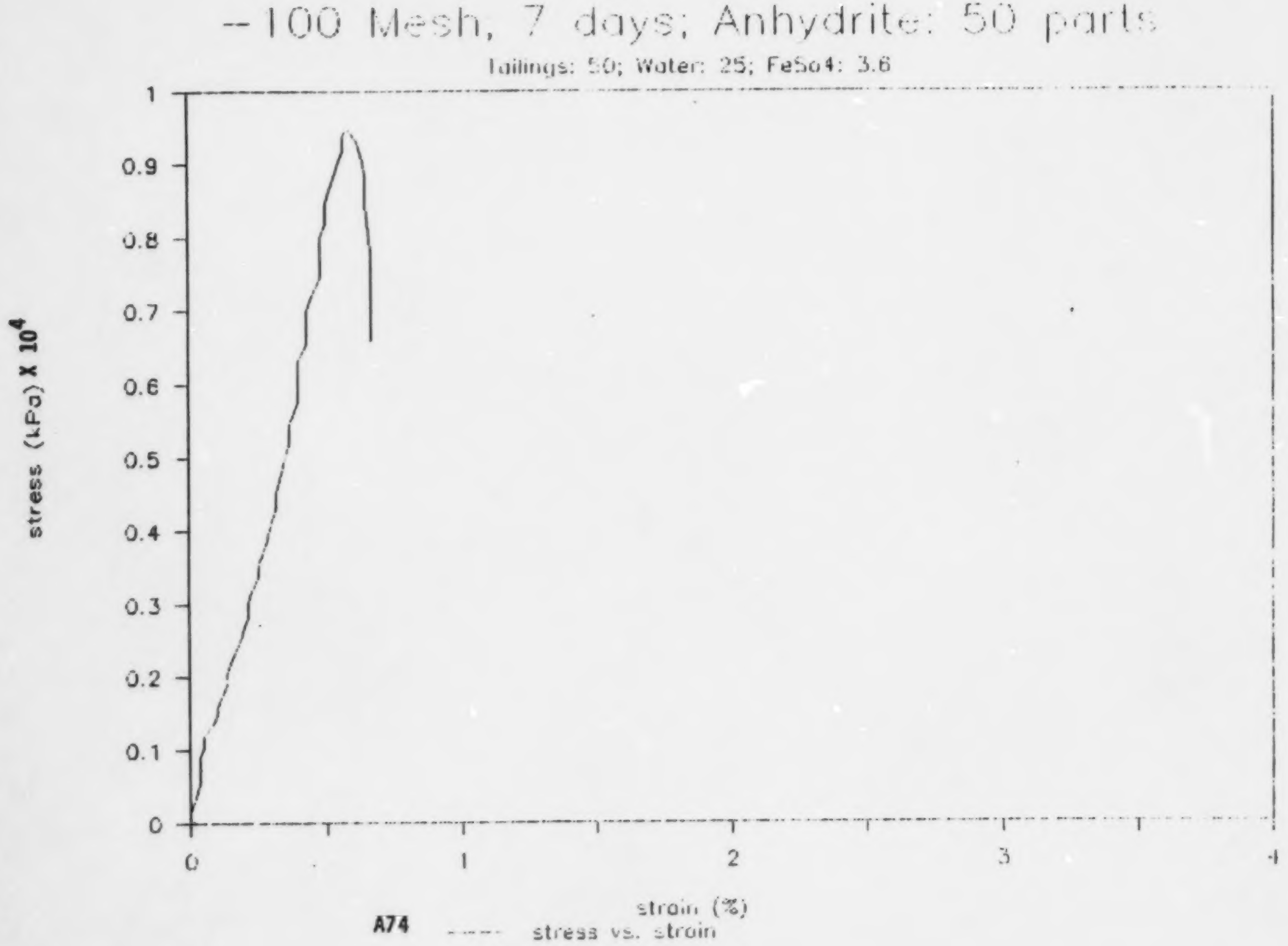
−100 Mesh, 7 days; Anhydrite: 25 parts

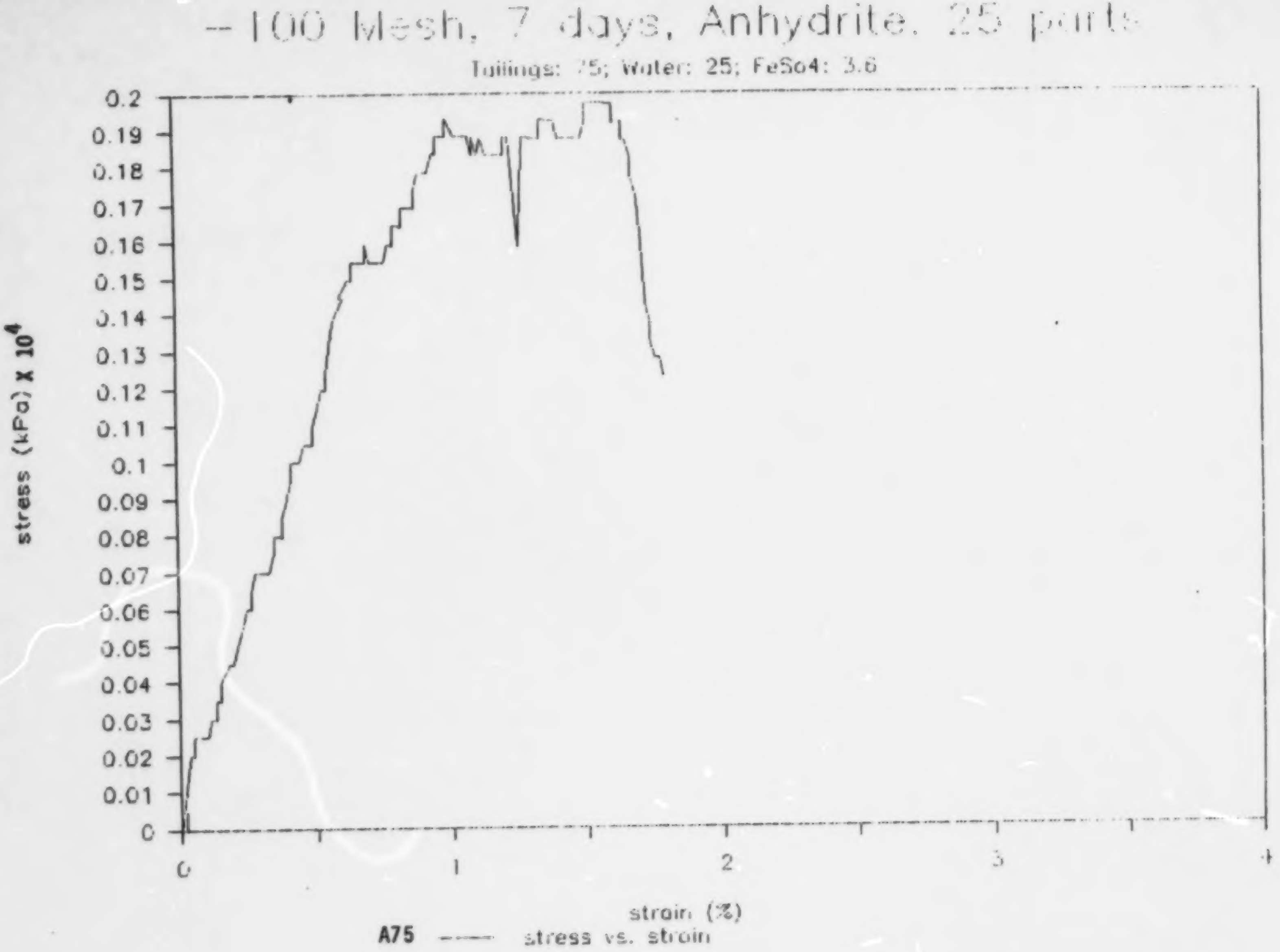
Tailings: 75; Water: 35; FeSo4: 2.8




-100 Mesh, 7 days; Anhydrite: 100 parts
Tailings: 0; Water: 25; FeSo4: 3.6
stress (kPa) X 10^4
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A72
stress vs. strain

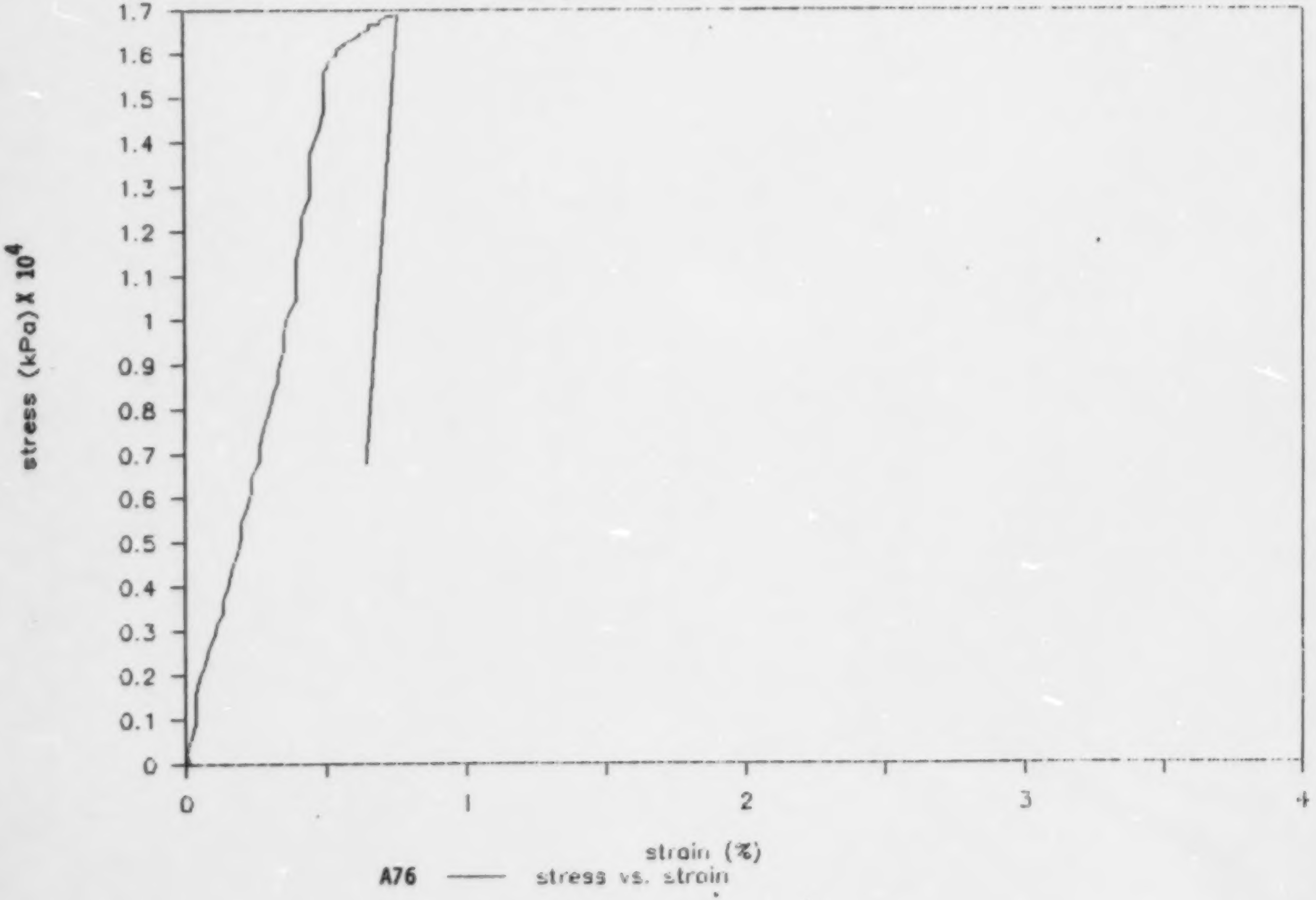

-100 Mesh, 7 days; Anhydrite: 75; [illegible]
Tailings: 25; Water: 25; FeSO4: 3.6
stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A73 —— stress vs. strain


–100 Mesh, 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 25; FeSo4: 3.6
stress (kPa) X 10^4
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A74
----- stress vs. strain


-100 Mesh, 7 days, Anhydrite. 25 parts
Tailings: 75; Water: 25; FeSo4: 3.6
stress (kPa) X 10^4
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A75
stress vs. strain

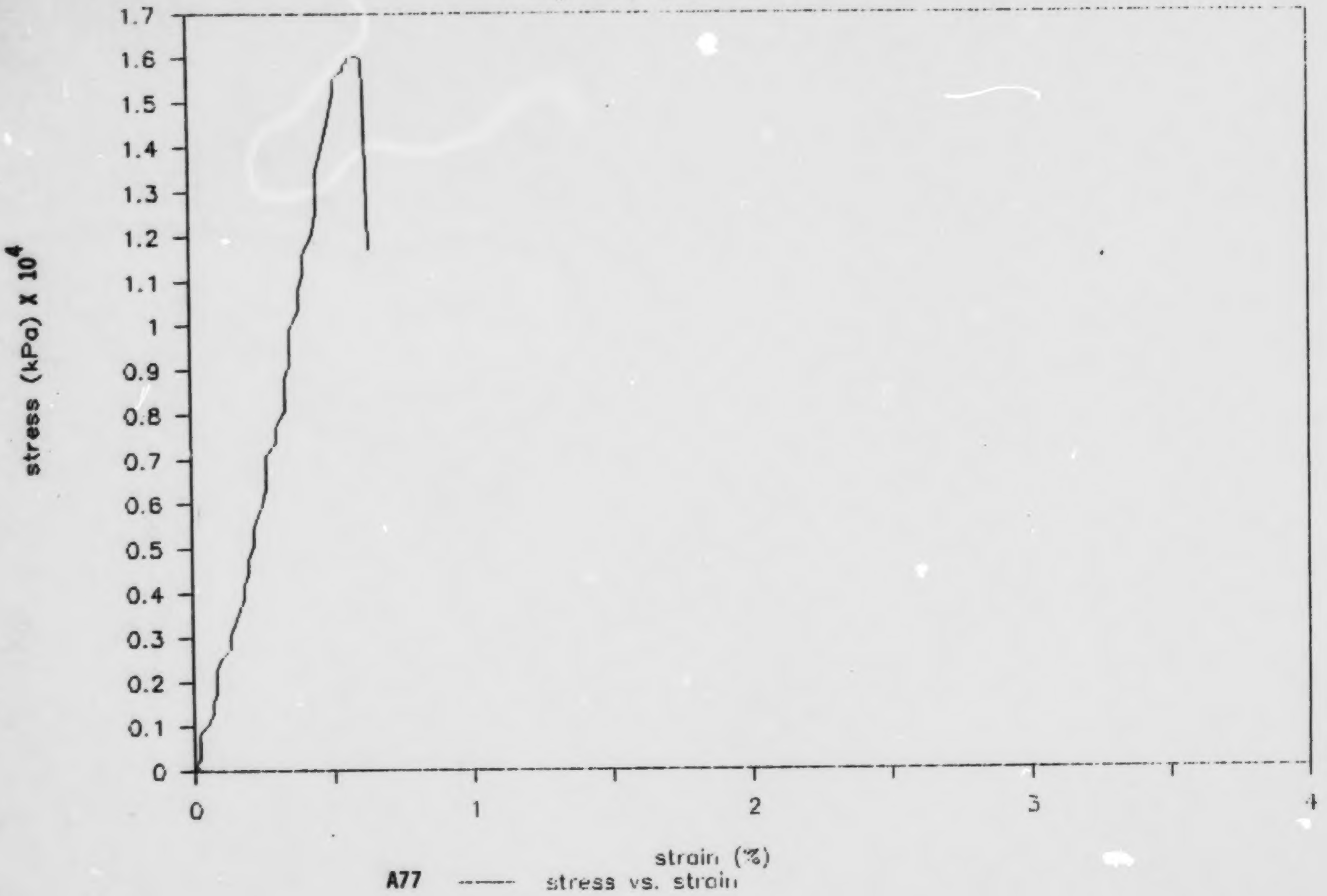
-100 Mesh; 7 days; Anhydrite. 100 parts

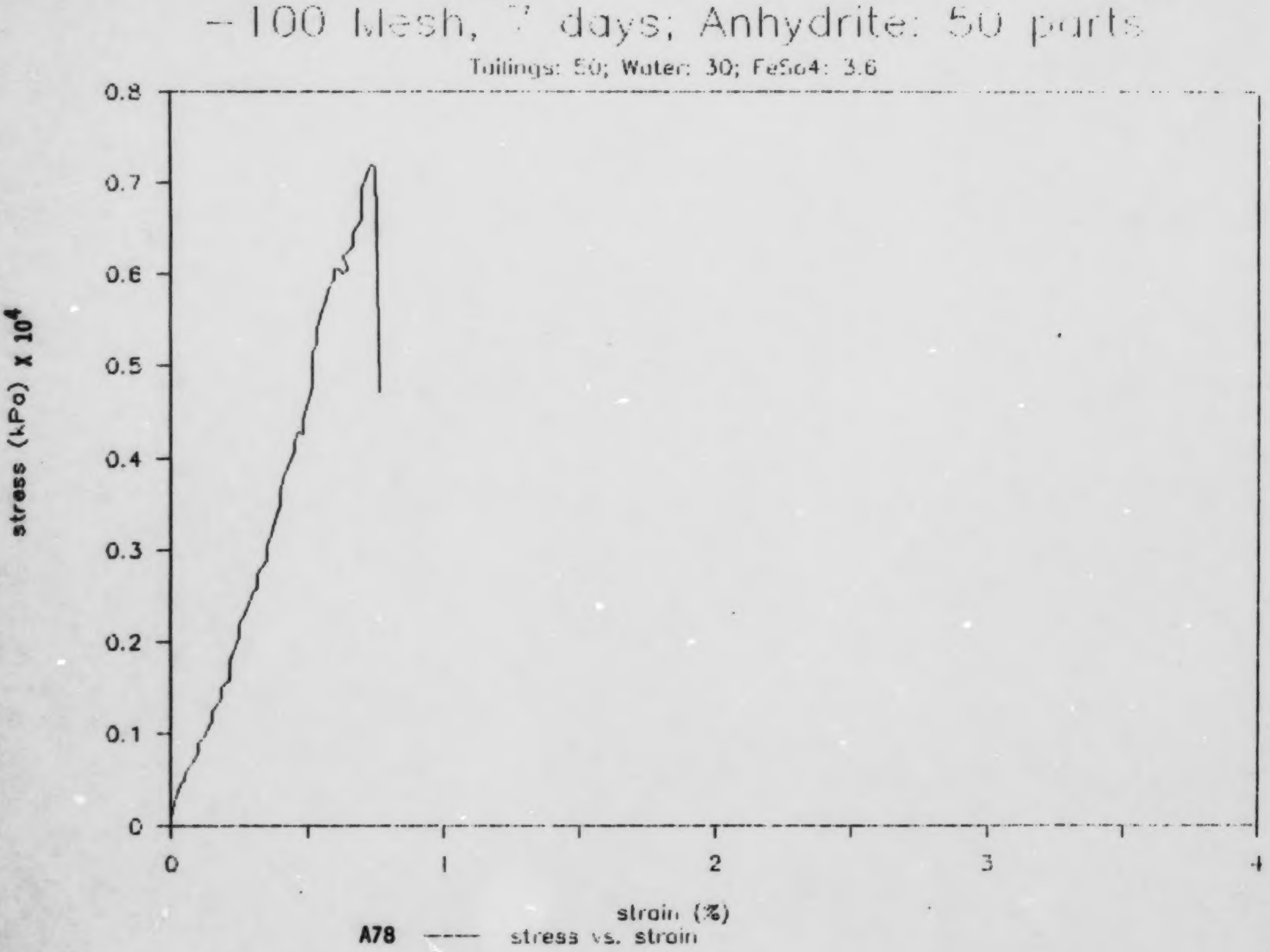
Tailings: 0; Water: 30; FeSo4: 3.6

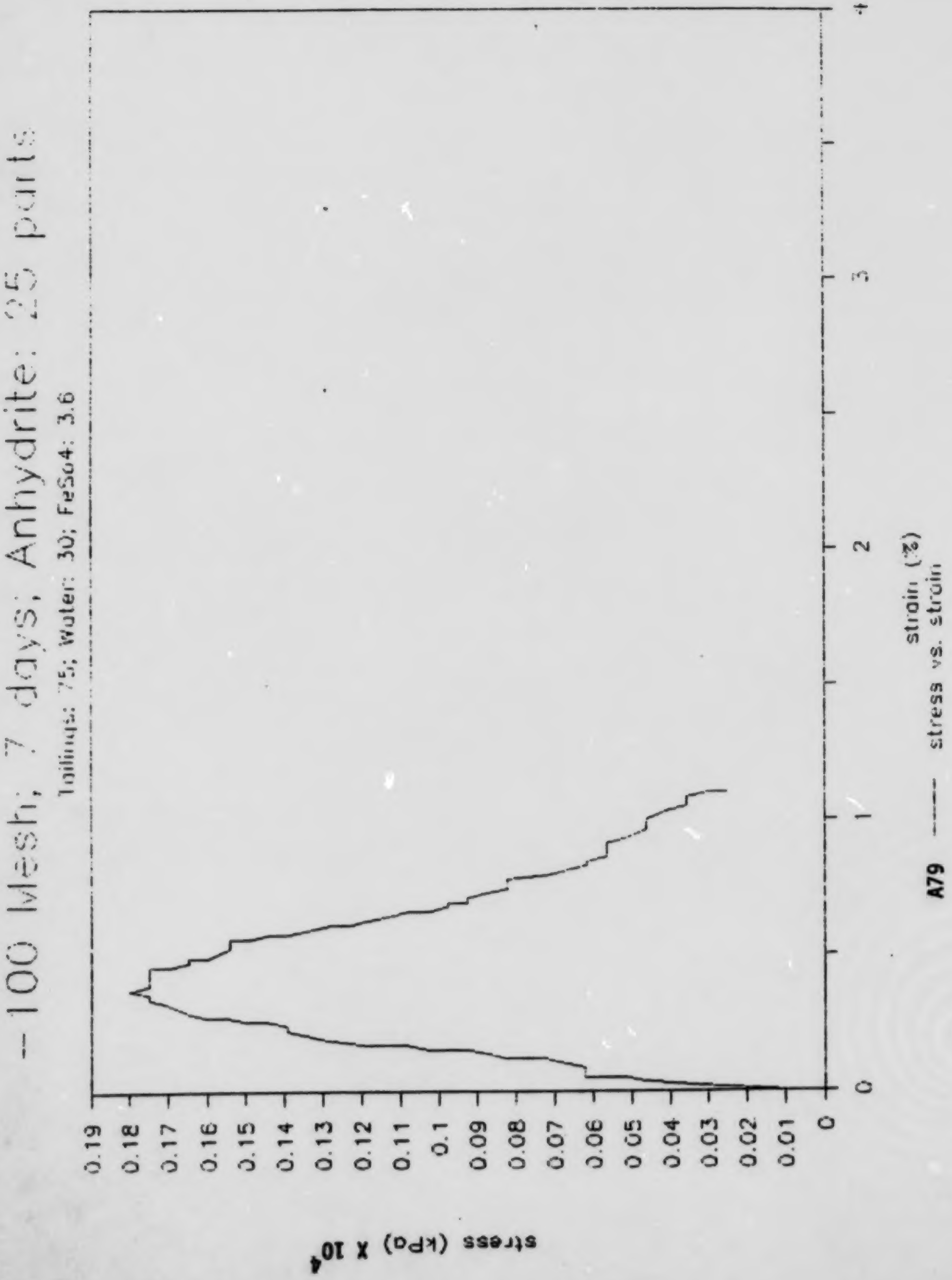


--100 Mesh; 7 days; Anhydrite: 75 parts .

Tailings: 25; Water: 30; FeSo4: 3.6

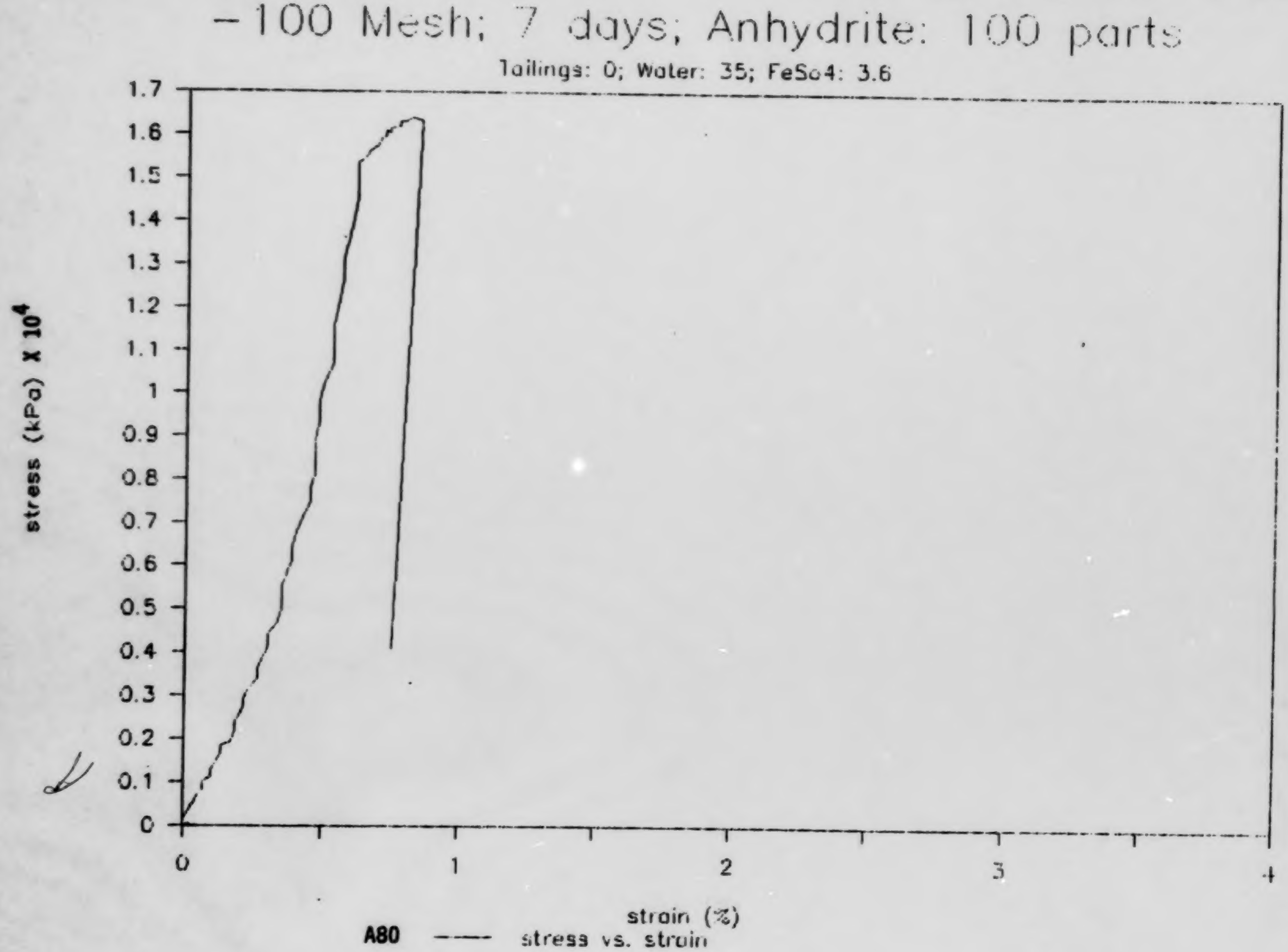


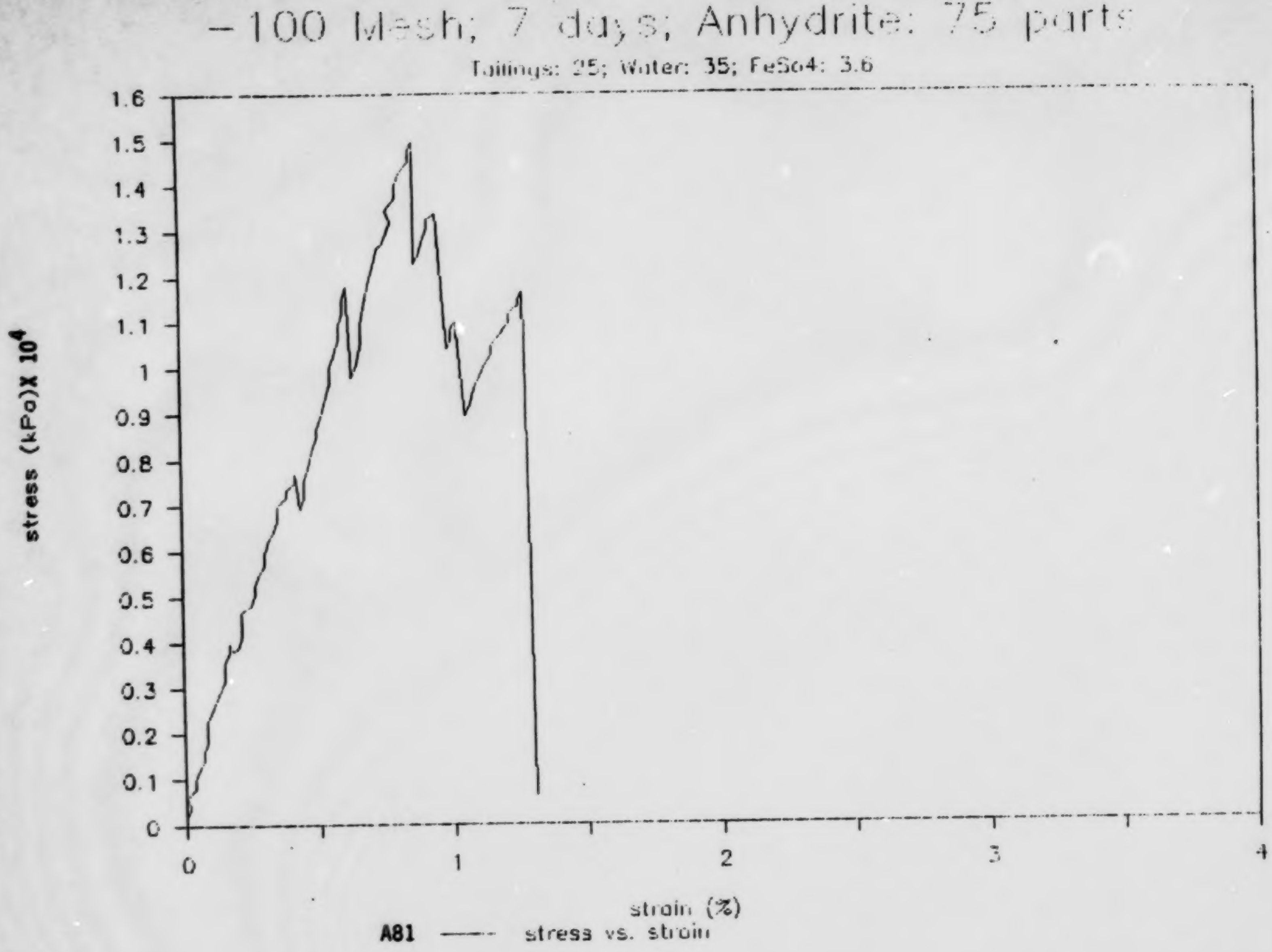

-100 Mesh, 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 30; FeSo4: 3.6
stress (kPa) X 10⁴
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A78 ---- stress vs. strain

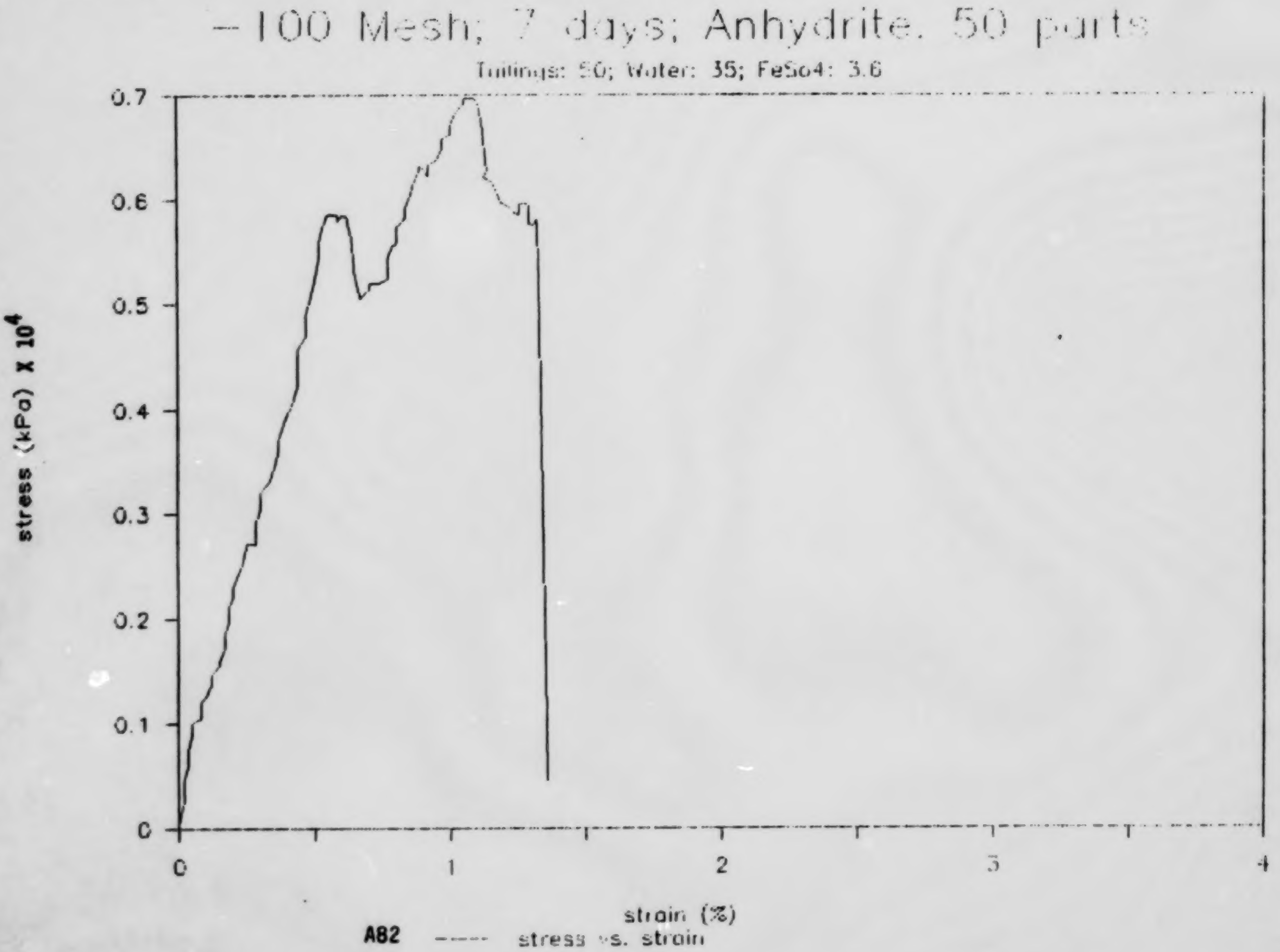


-100 Mesh; 7 days; Anhydrite: 25 parts

Tailings: 75; Water: 30; FeSo4: 3.6

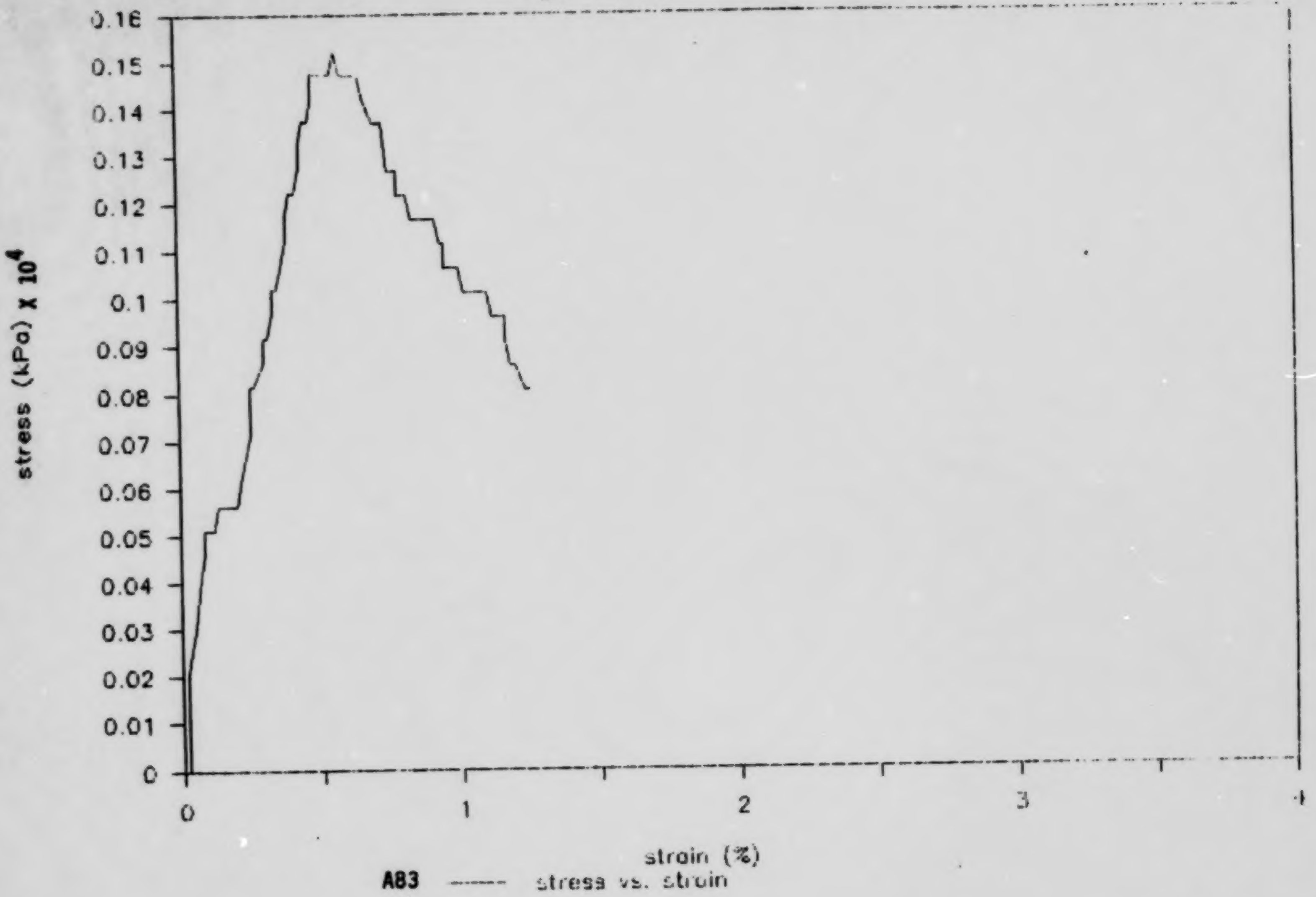

−100 Mesh; 7 days; Anhydrite: 100 parts
Tailings: 0; Water: 35; FeSo4: 3.6
stress (kPa) X 10⁴
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A80
stress vs. strain


-100 Mesh; 7 days; Anhydrite: 75 parts
Tailings: 25; Water: 35; FeSo4: 3.6
stress (kPa)X 10⁴
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A81 —— stress vs. strain


−100 Mesh; 7 days; Anhydrite. 50 parts
Tailings: 50; Water: 35; FeSo4: 3.6
stress (kPa) X 10⁴
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A82 ---- stress vs. strain

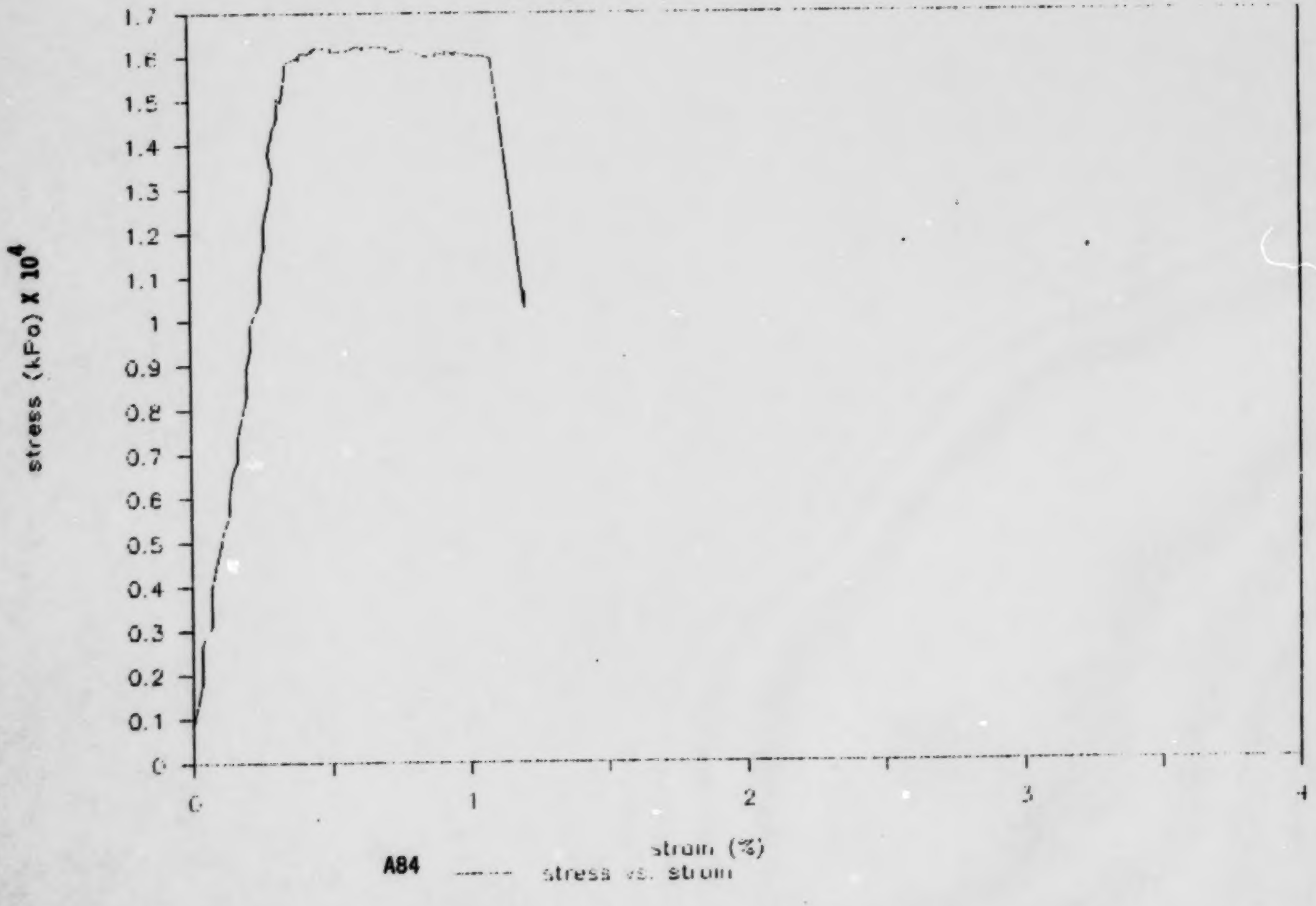
-100 Mesh, 7 days; Anhydrite: 25 parts

Tailings: 75; Water: 35; FeSo4: 3.6



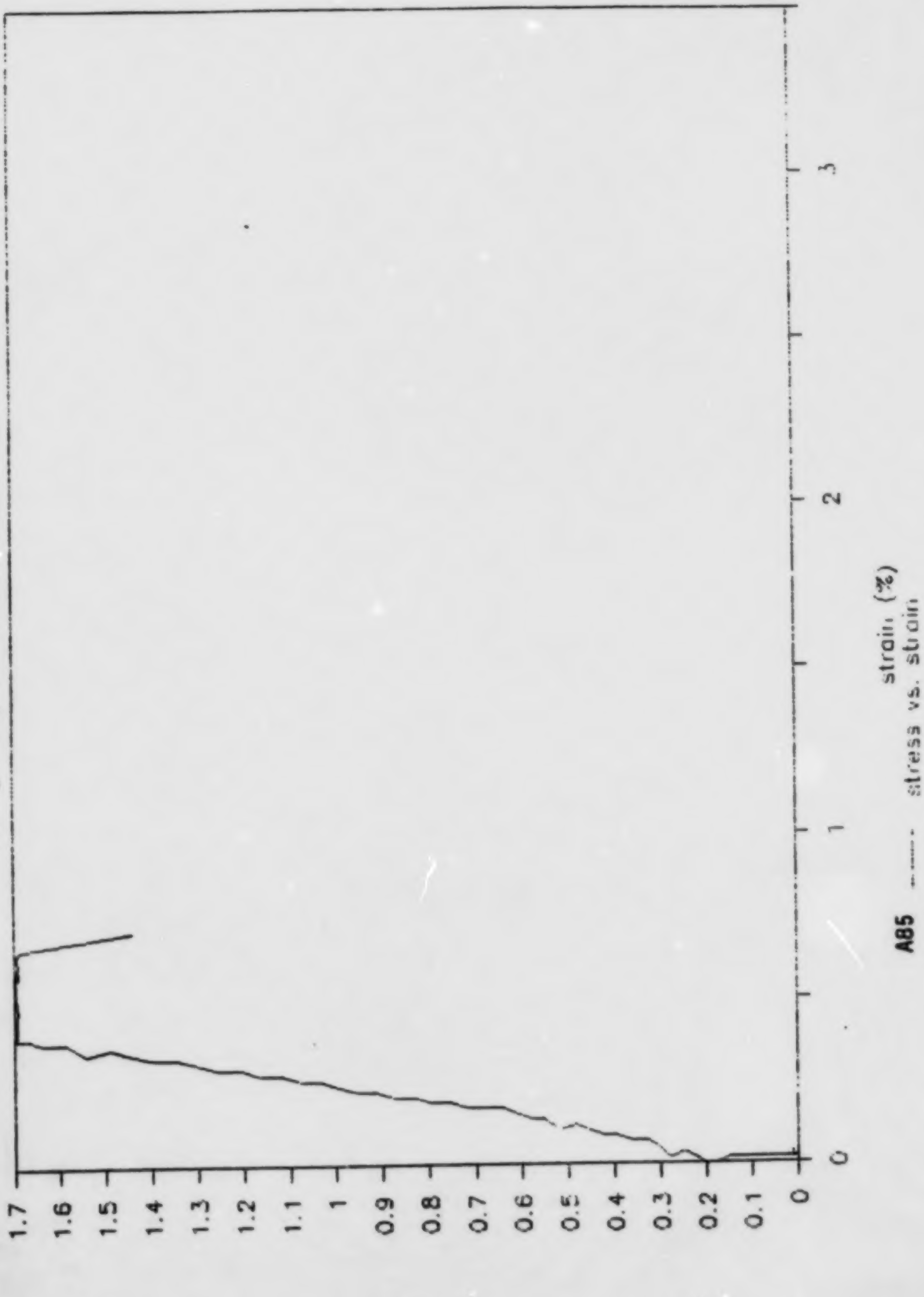
-100 Mesh; 28 days, Anhydrite 100part

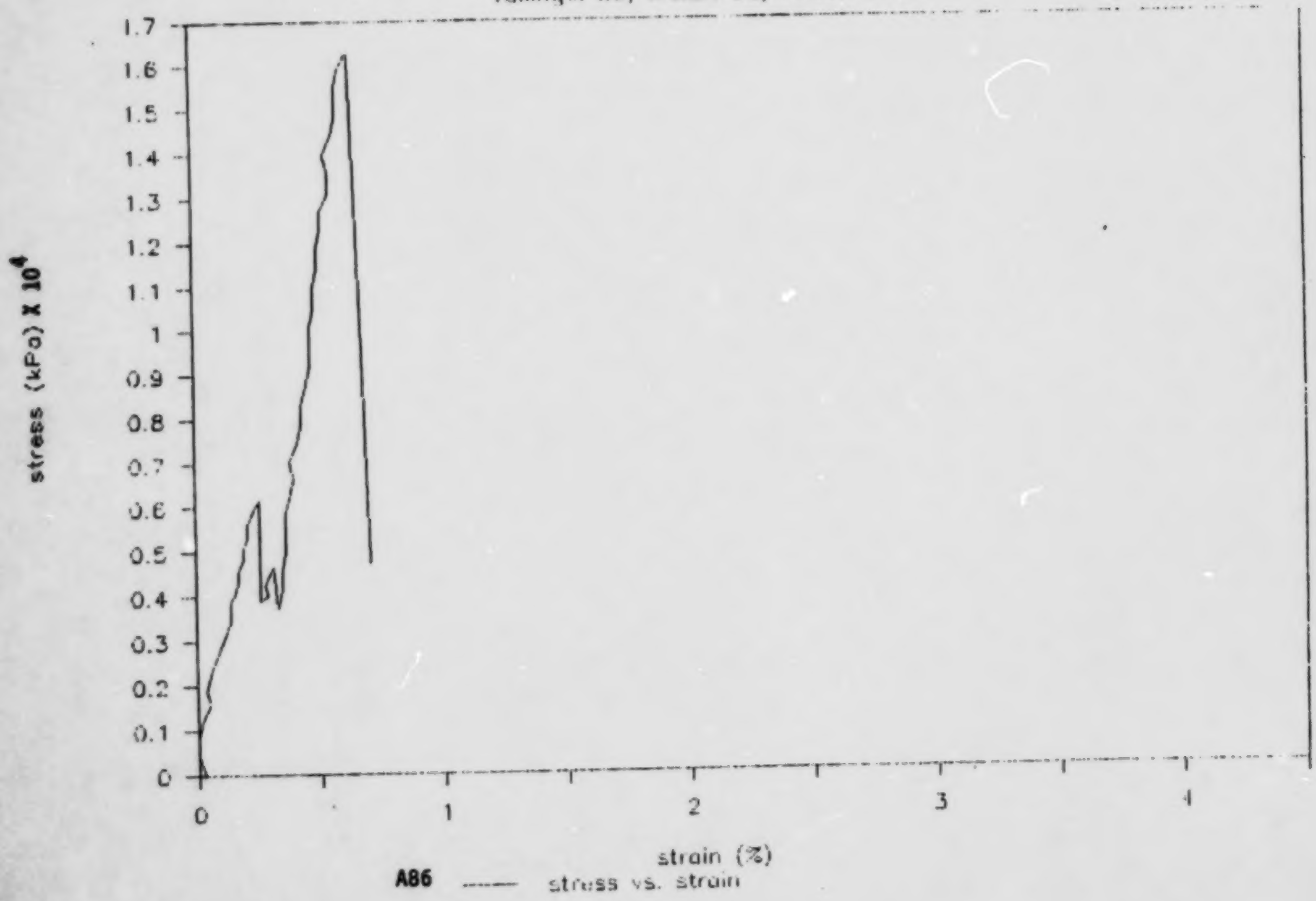
Tailings: 0; Water: 25; FeSO4: 2.8



-100; 28 days; Anhydrite. 75 parts

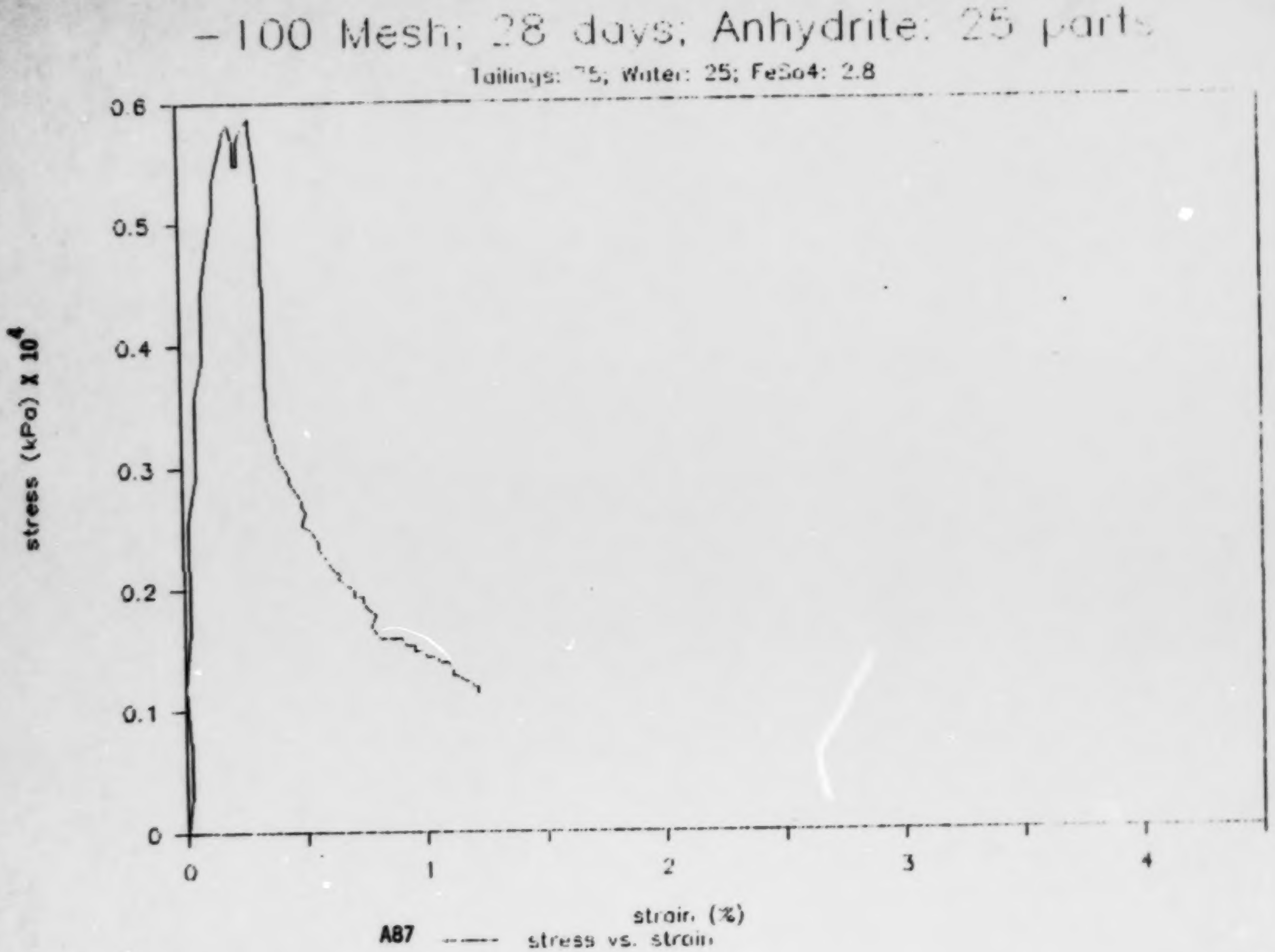
Tailings: 25; Water: 25; FeSO4: 2.8





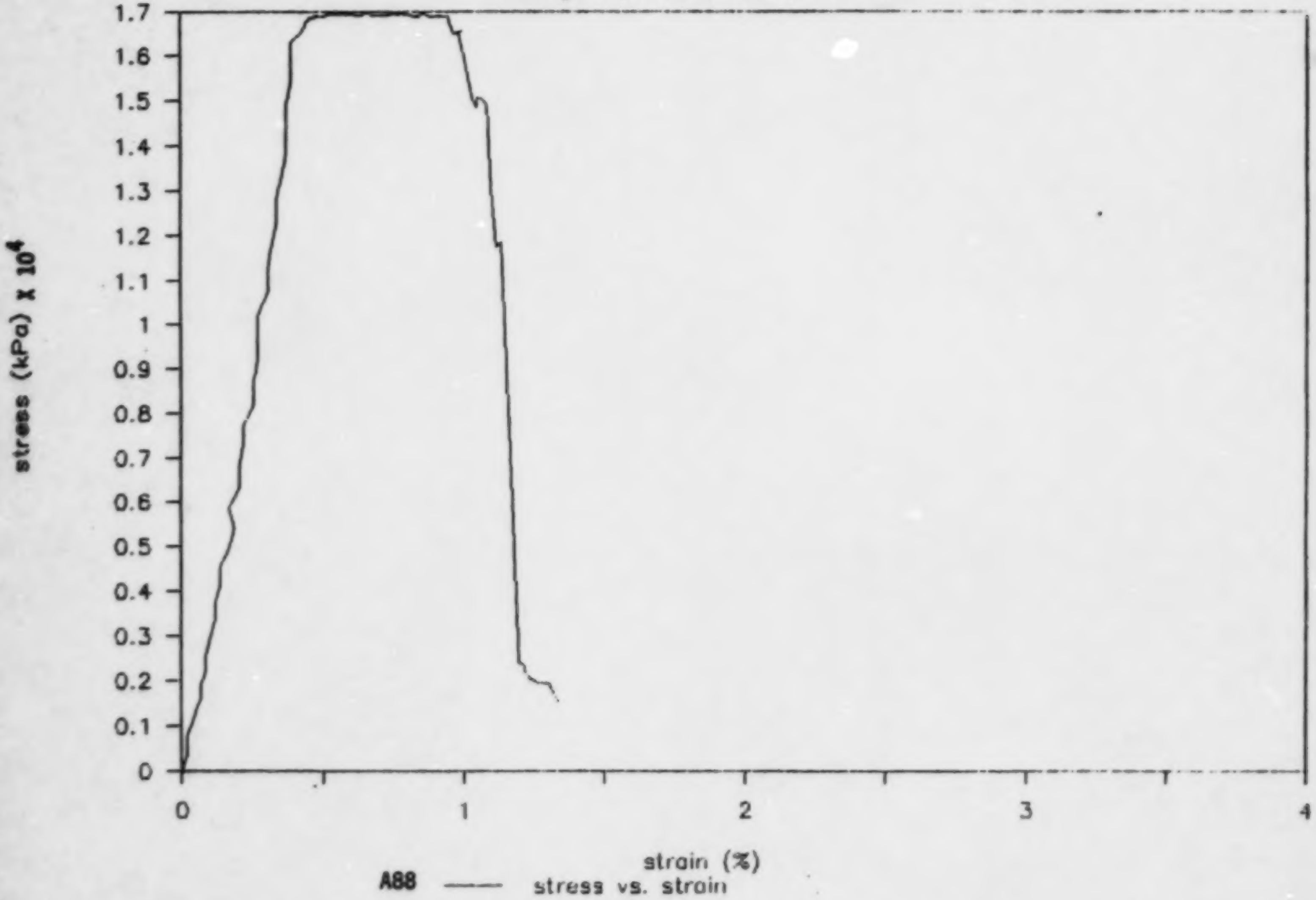
−100 Mesh; 28 days; Anhydrite: 50 parts

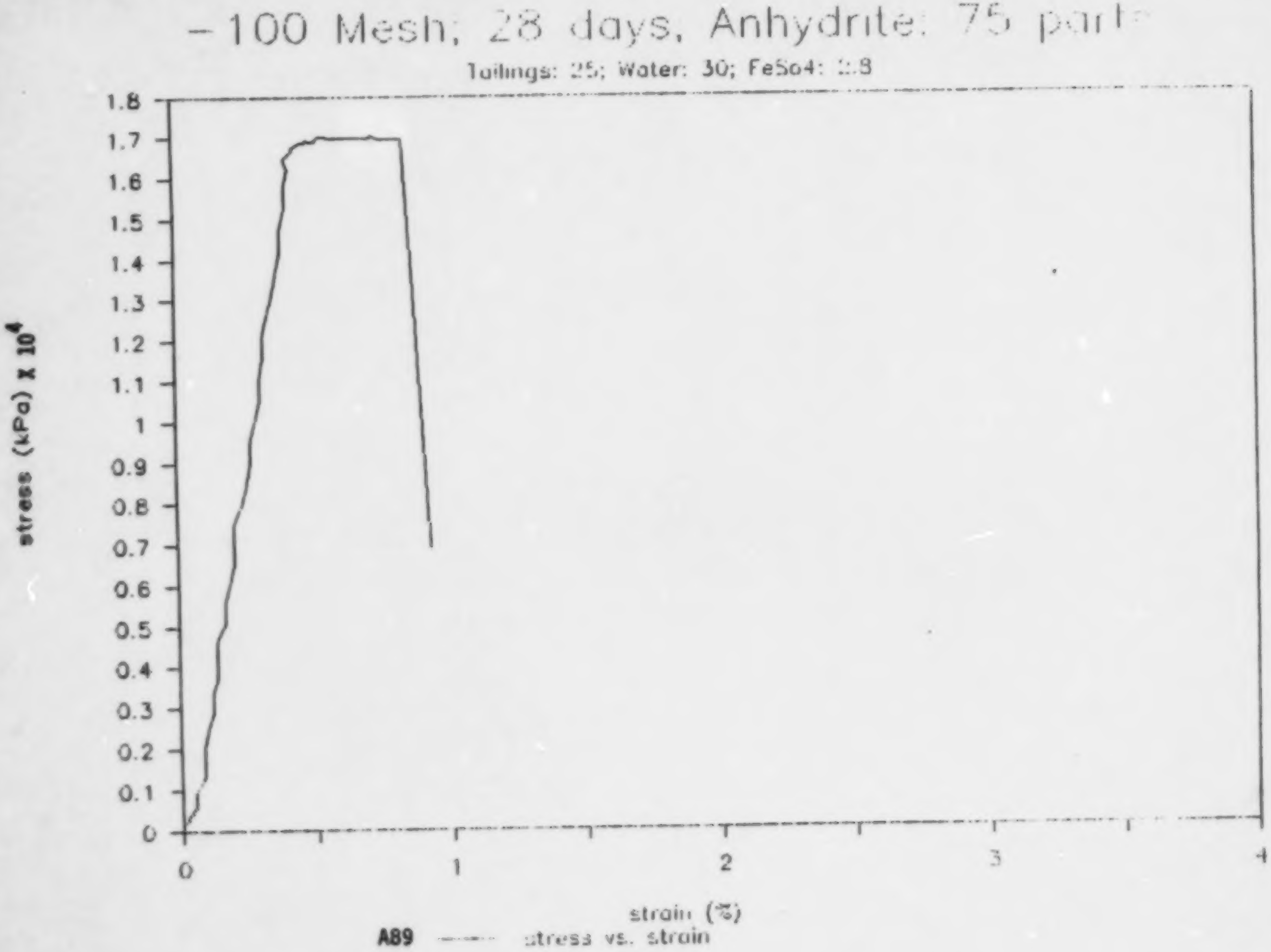
Tailings: 50; Water: 25; FeSo4: 2.9


−100 Mesh; 28 days; Anhydrite: 25 parts
Tailings: 75; Water: 25; FeSo4: 2.8
stress (kPa) X 10⁴
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A87
stress vs. strain

−100 Mesh; 28 days; Anhydrite: 100parts

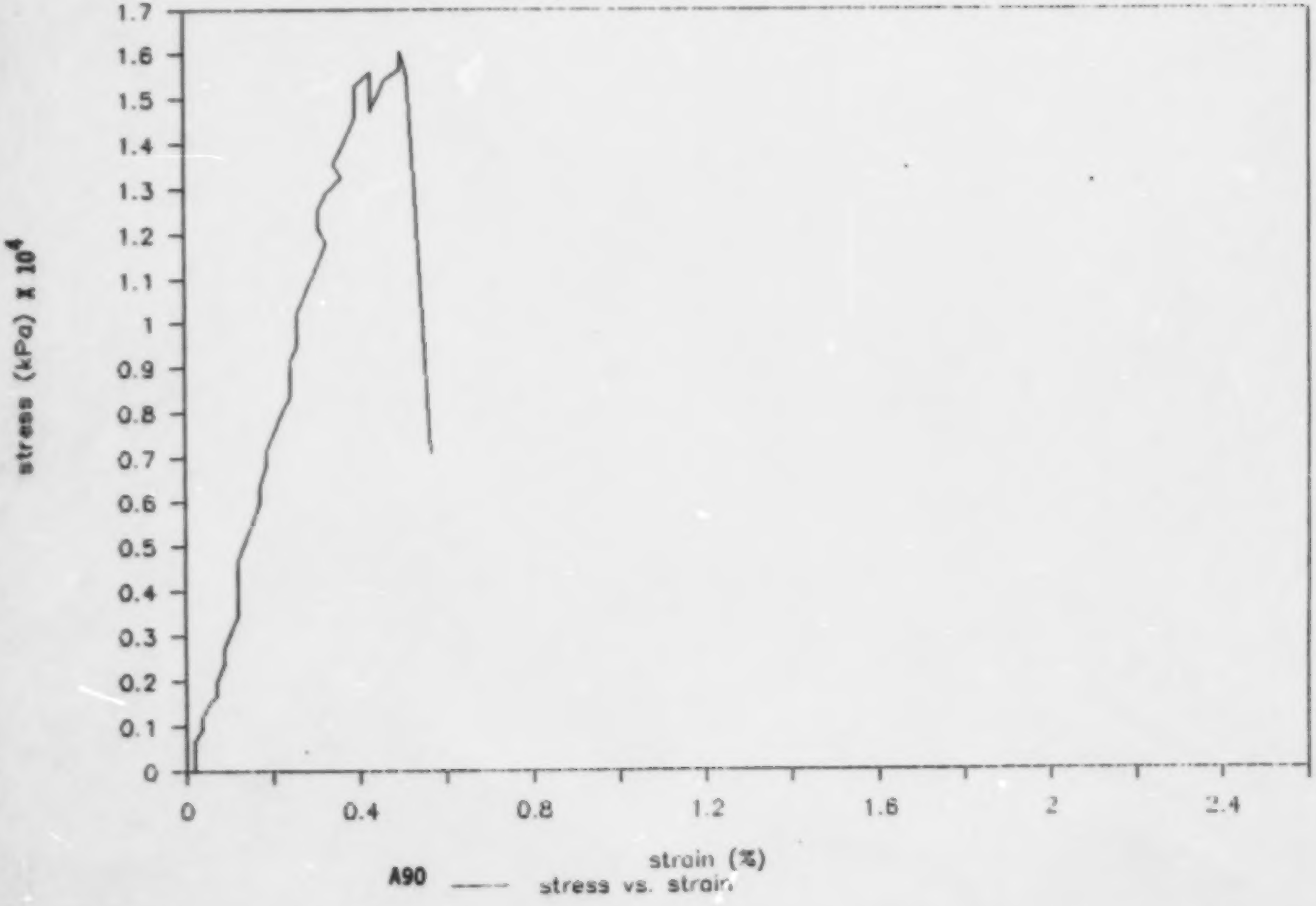
Tailings: 0; Water: 30; FeSo4: 2.8

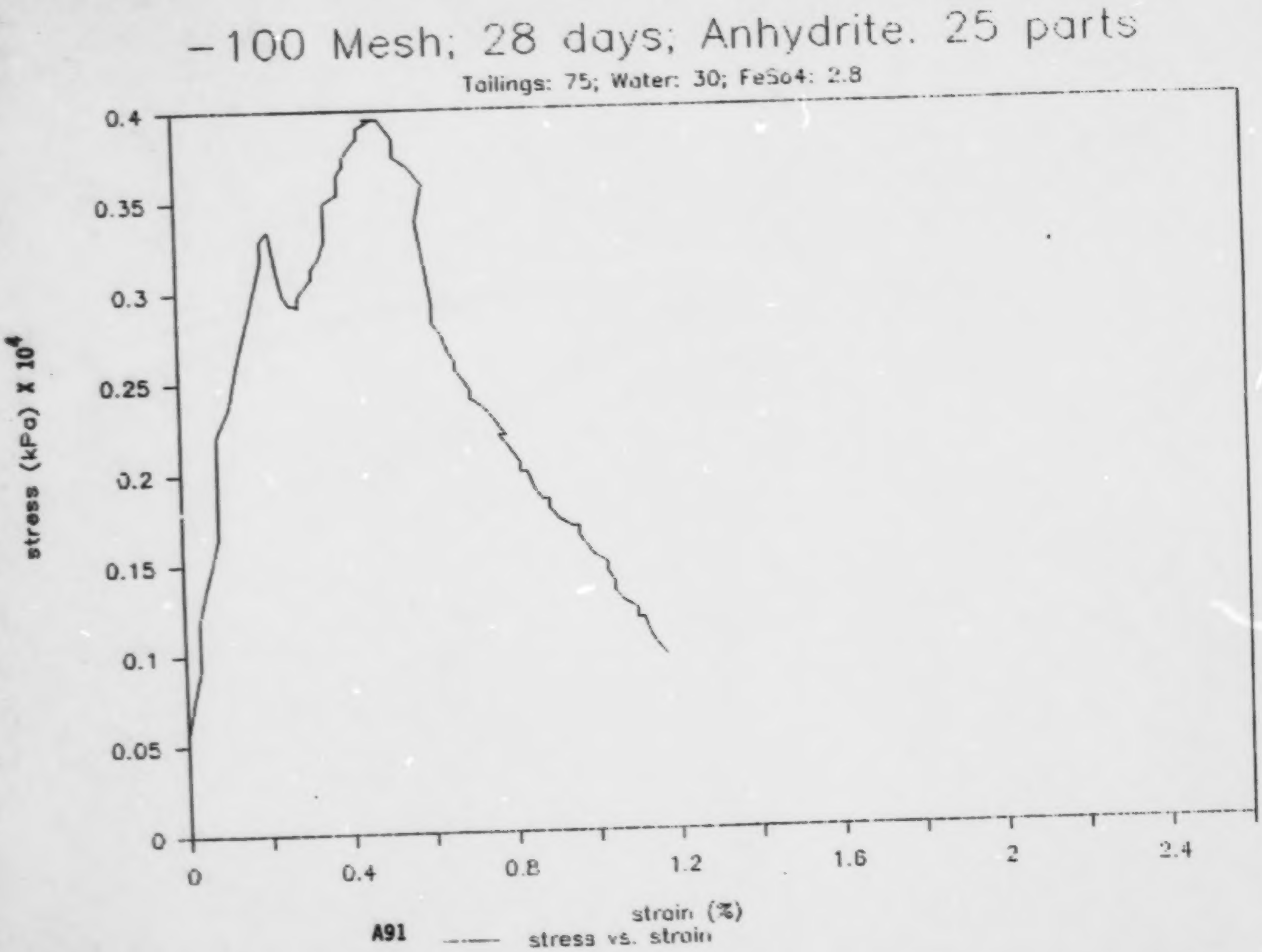


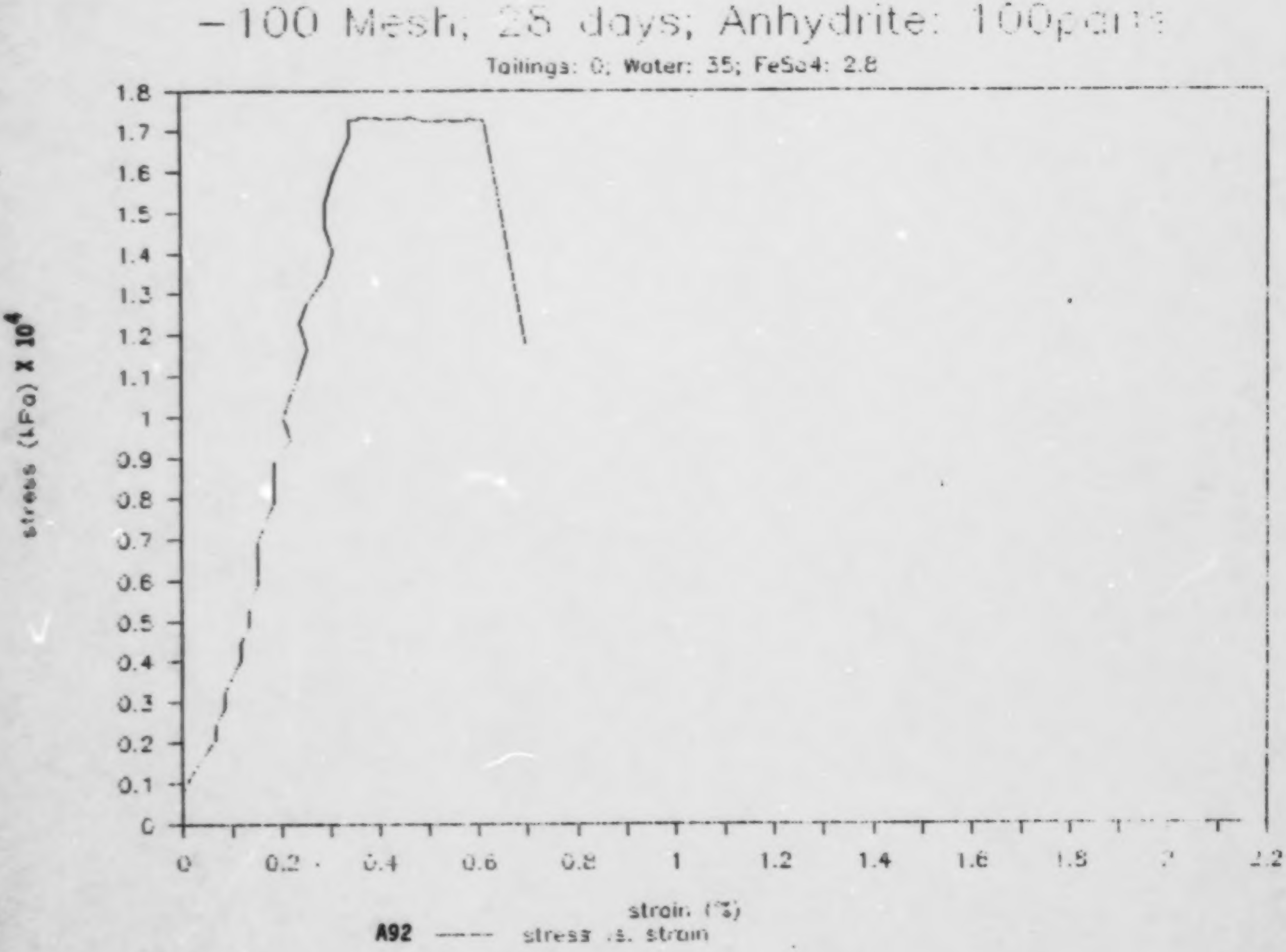

−100 Mesh; 28 days, Anhydrite: 75 parts
Tailings: 25; Water: 30; FeSo4: 2.8
stress (kPa) X 10⁴
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A89
stress vs. strain

−100 Mesh; 28 days; Anhydrite: 50 parts

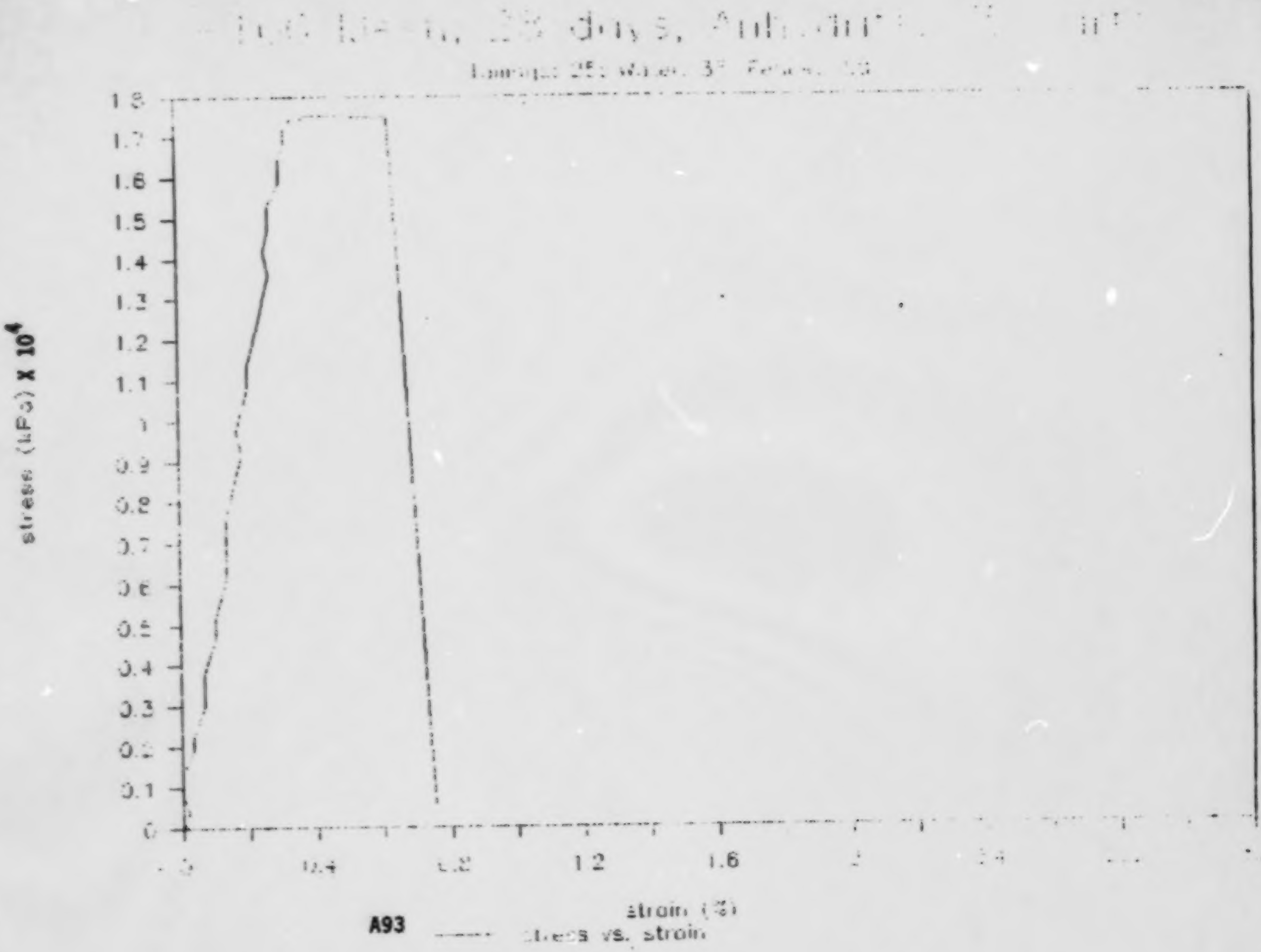
Tailings: 50; Water: 30; FeSo4: 2.8

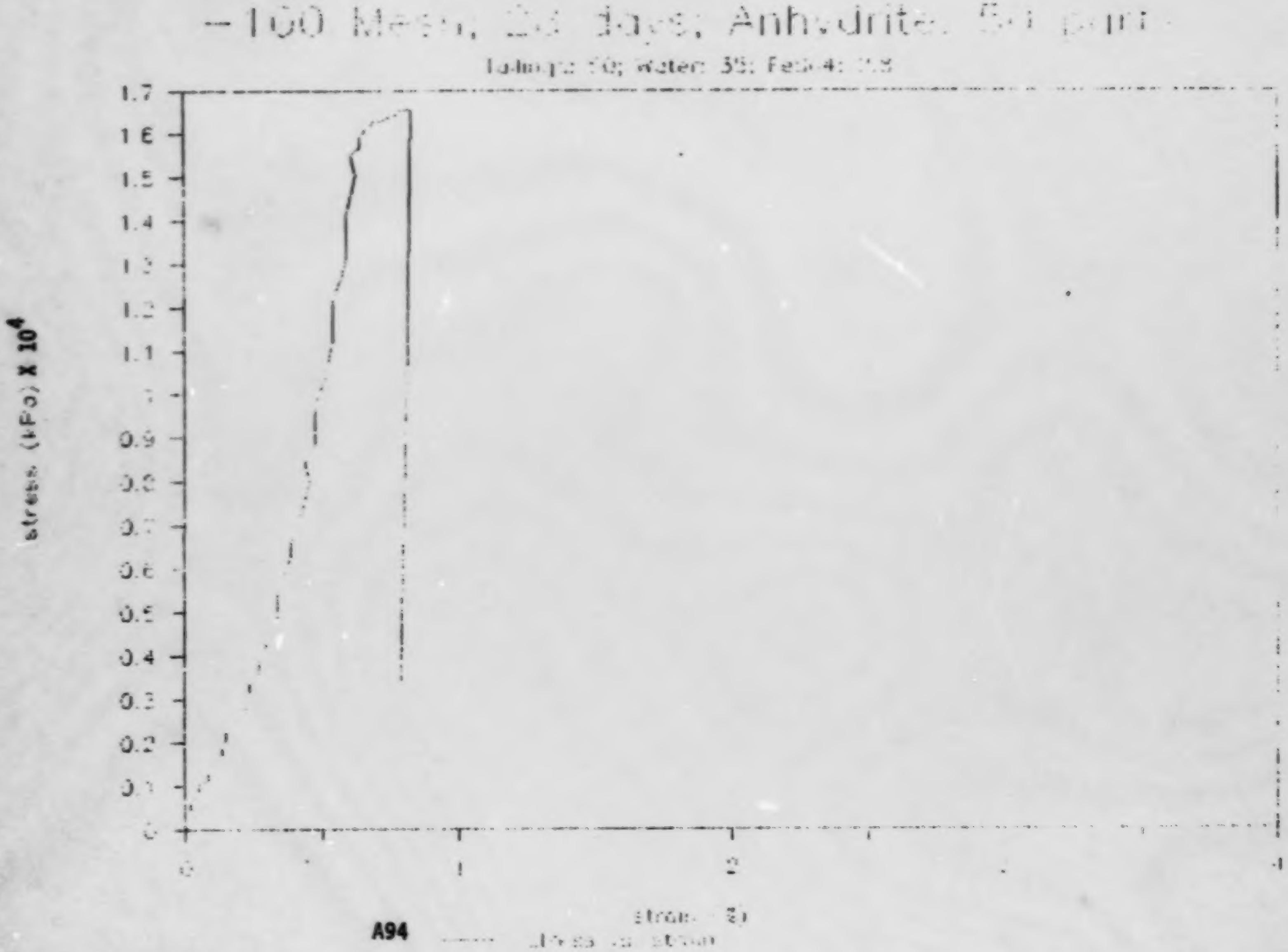



−100 Mesh; 28 days; Anhydrite. 25 parts
Tailings: 75; Water: 30; FeSo4: 2.8
stress (kPa) X 10⁴
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
0.4
0.8
1.2
1.6
2
2.4
strain (%)
A91
stress vs. strain

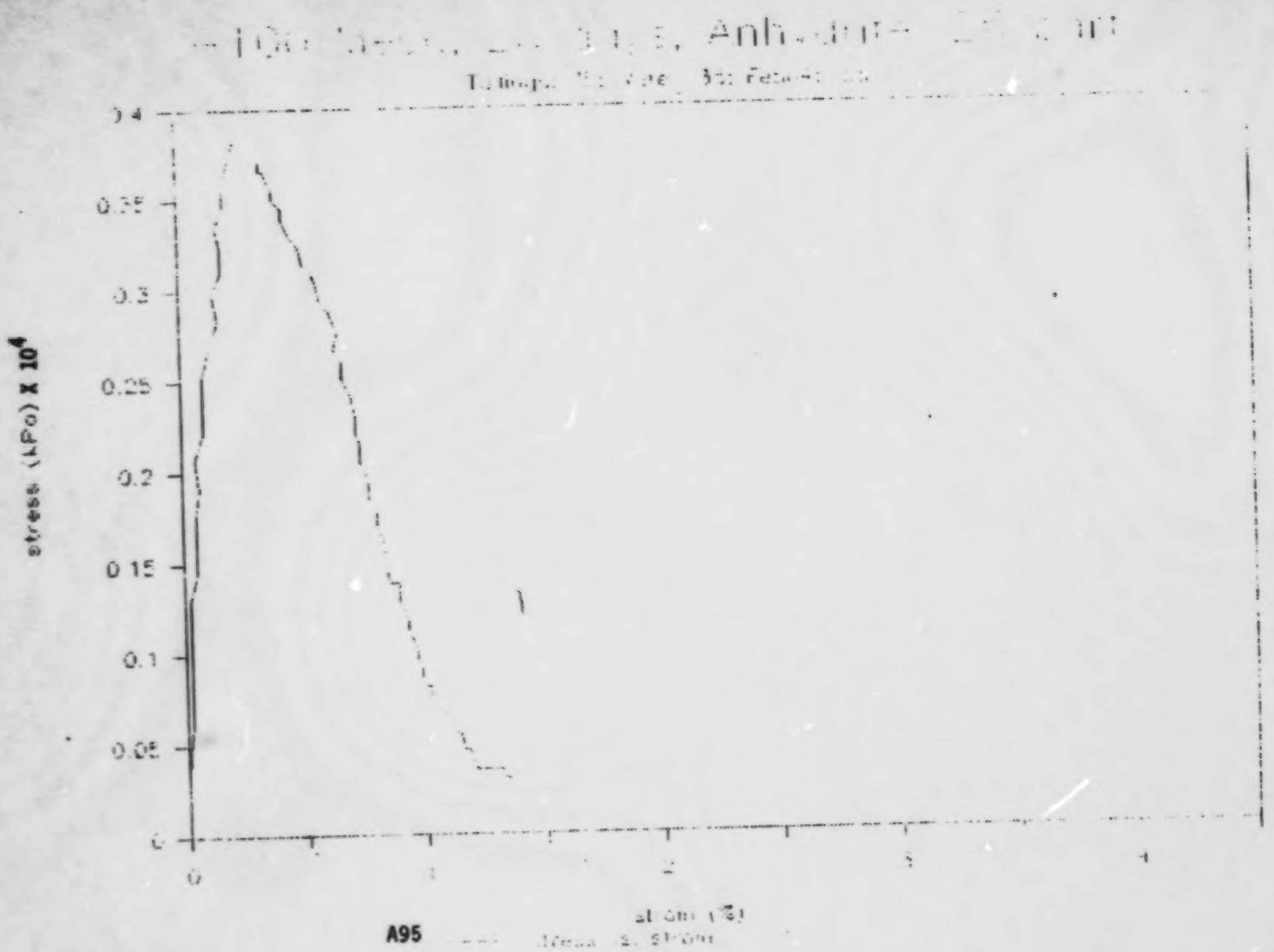


−100 Mesh, 28 days; Anhydrite: 100parts
Tailings: 0; Water: 35; FeSo4: 2.8
stress (kPa) X 10⁴
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
strain (%)
A92 ---- stress vs. strain

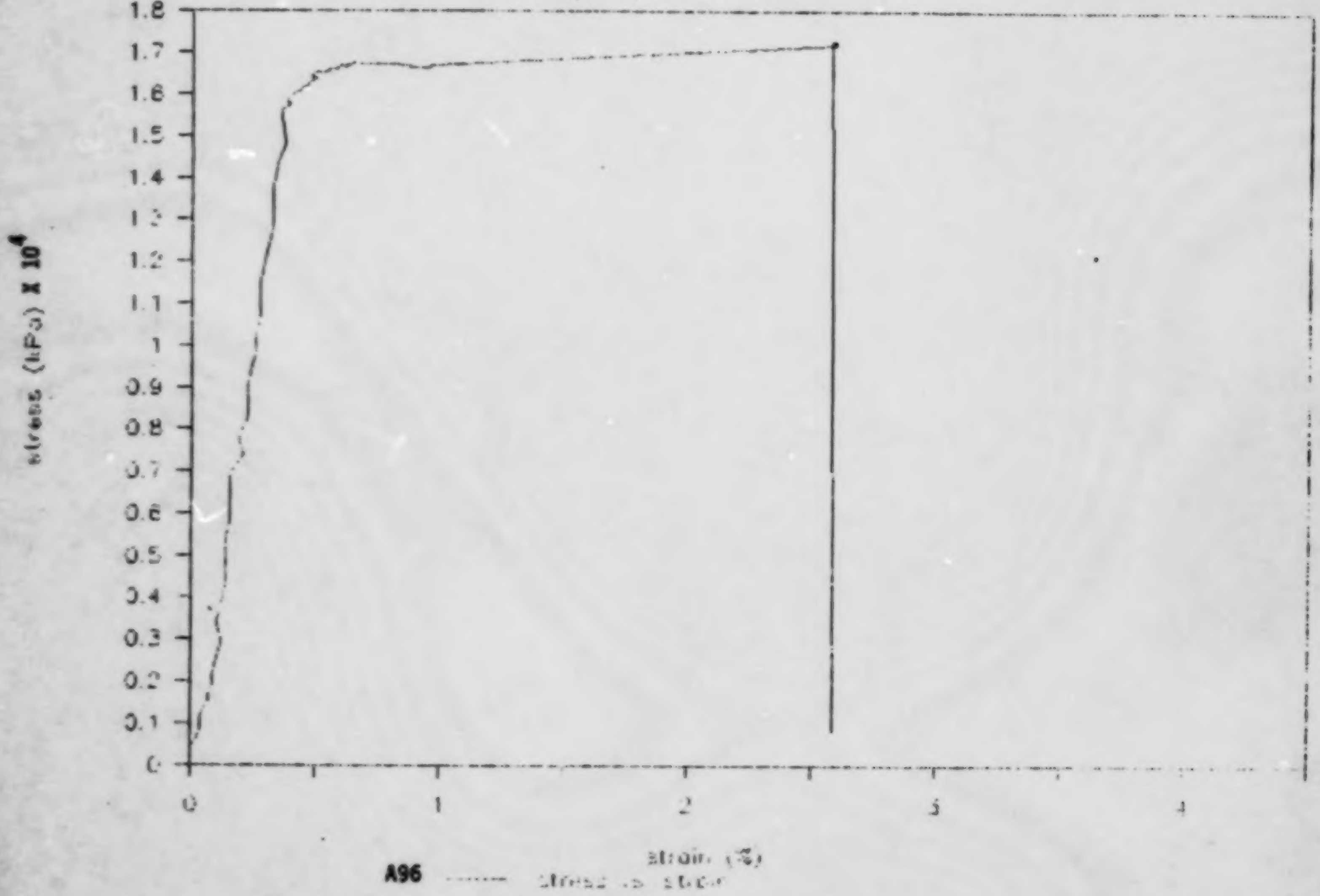

stress (kPa) X 10⁴
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.4
0.8
1.2
1.6
2
2.4
strain (%)
A93
stress vs. strain


−100 Mesh, 28 days; Anhydrite: 5 ppm
Tailings: 60; Water: 35; FeSO4: 5
stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
4
strain (%)
stress vs strain

A94

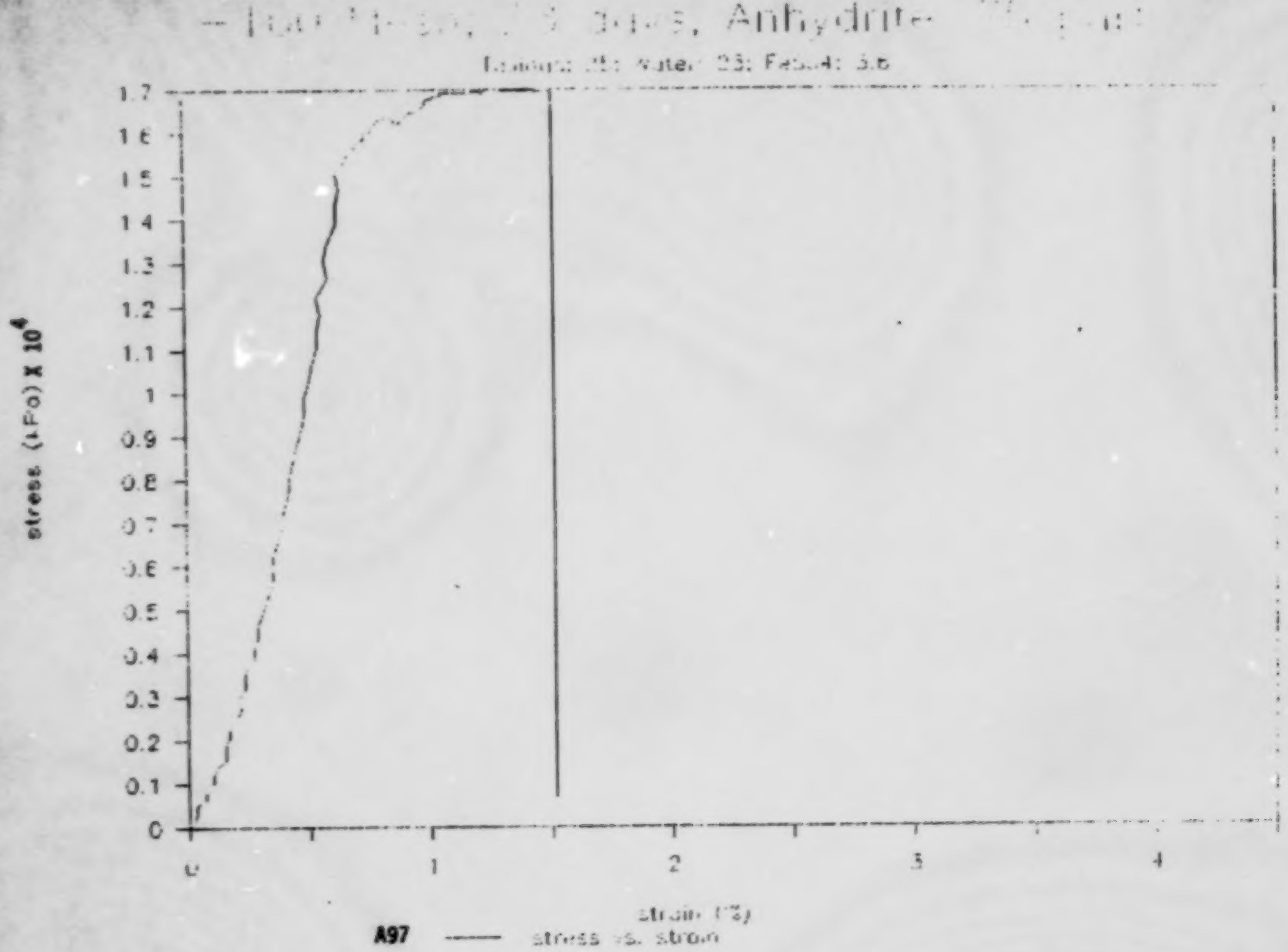

Anhydrite
stress (kPa) X 10⁴
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
3
4
strain (%)
stress vs. strain

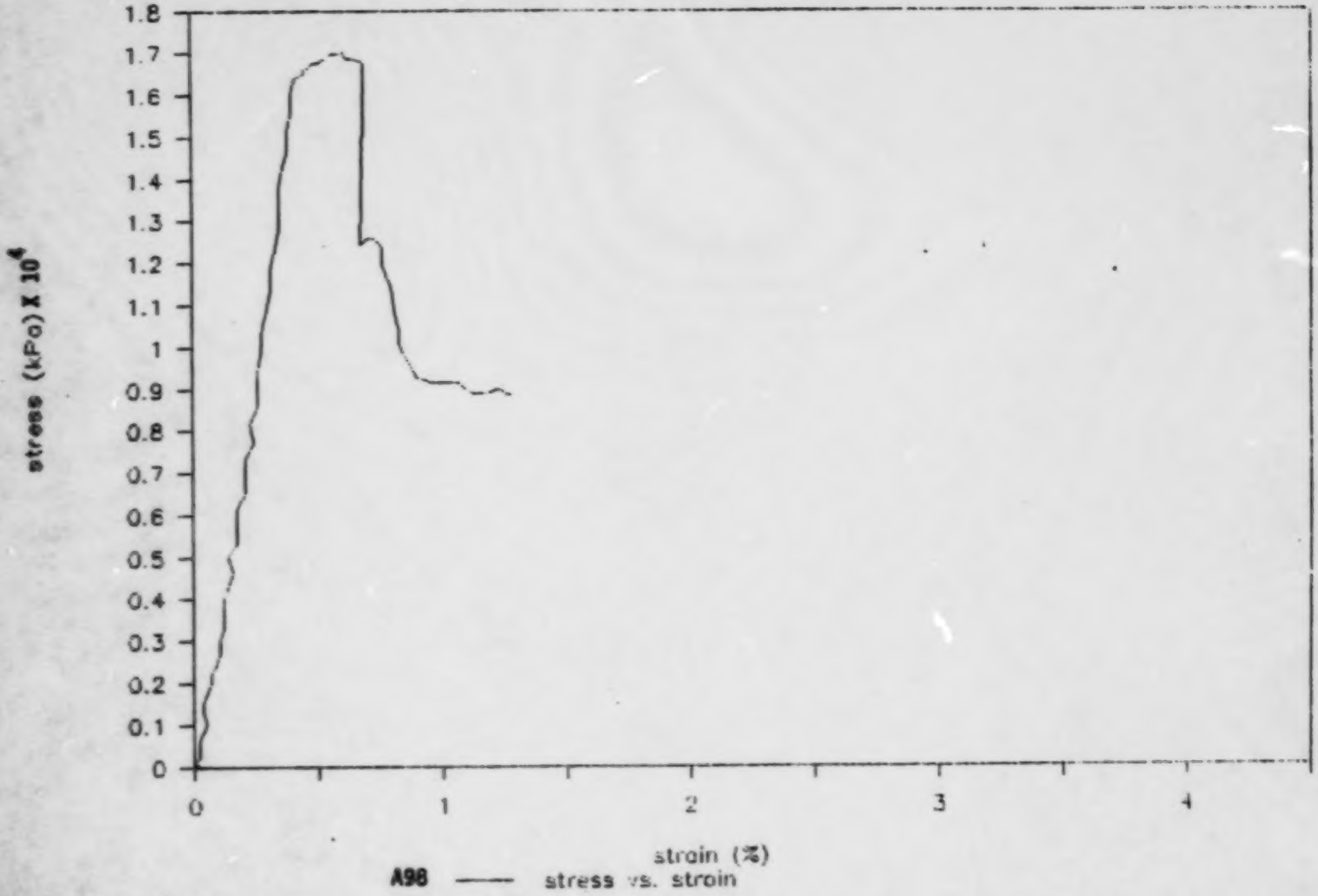
A95



−100 Mesh, 28 days, Anhydrite [illegible]

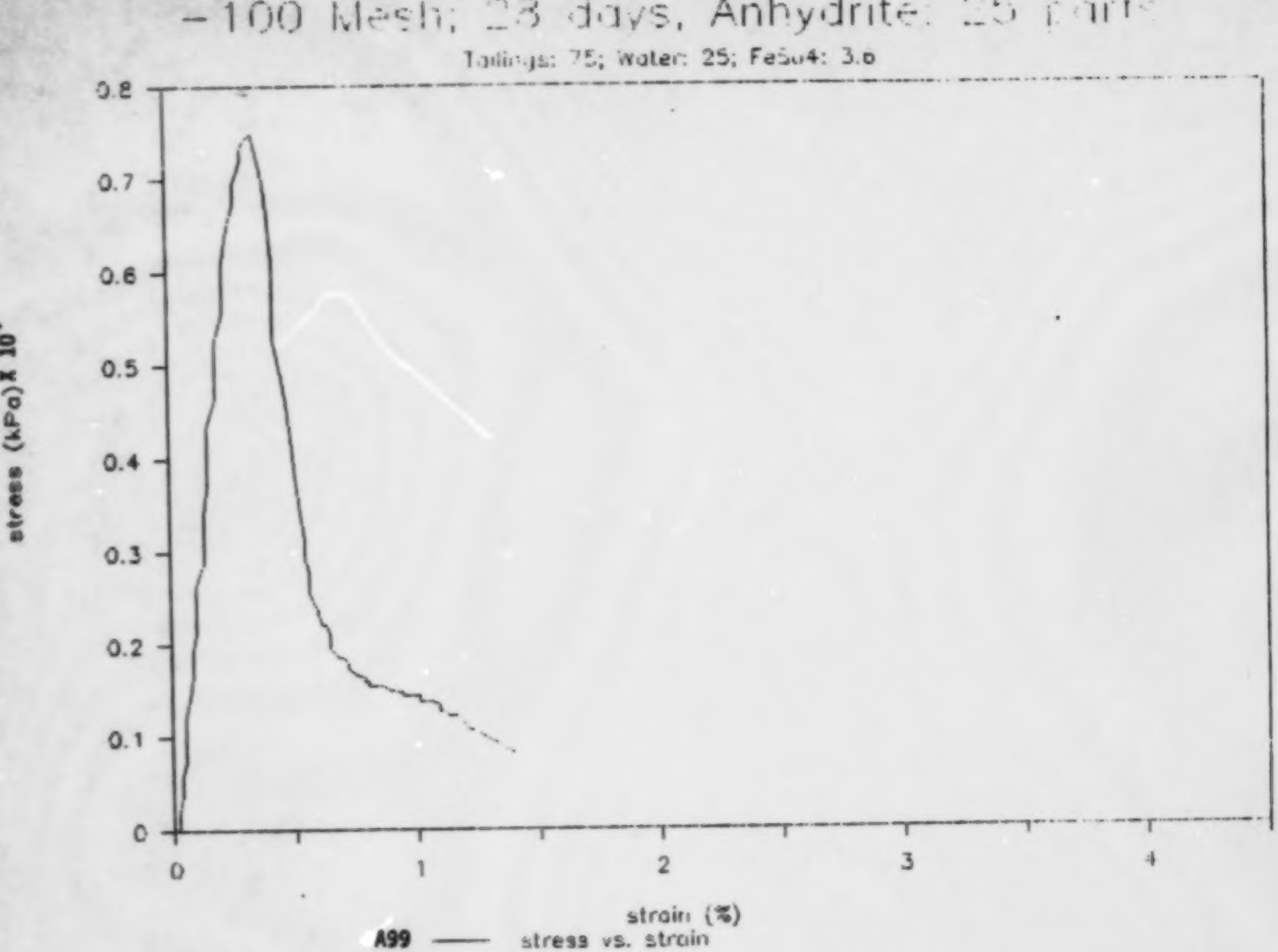
Tailings: 6; water: 25; FeSo4: 3.6

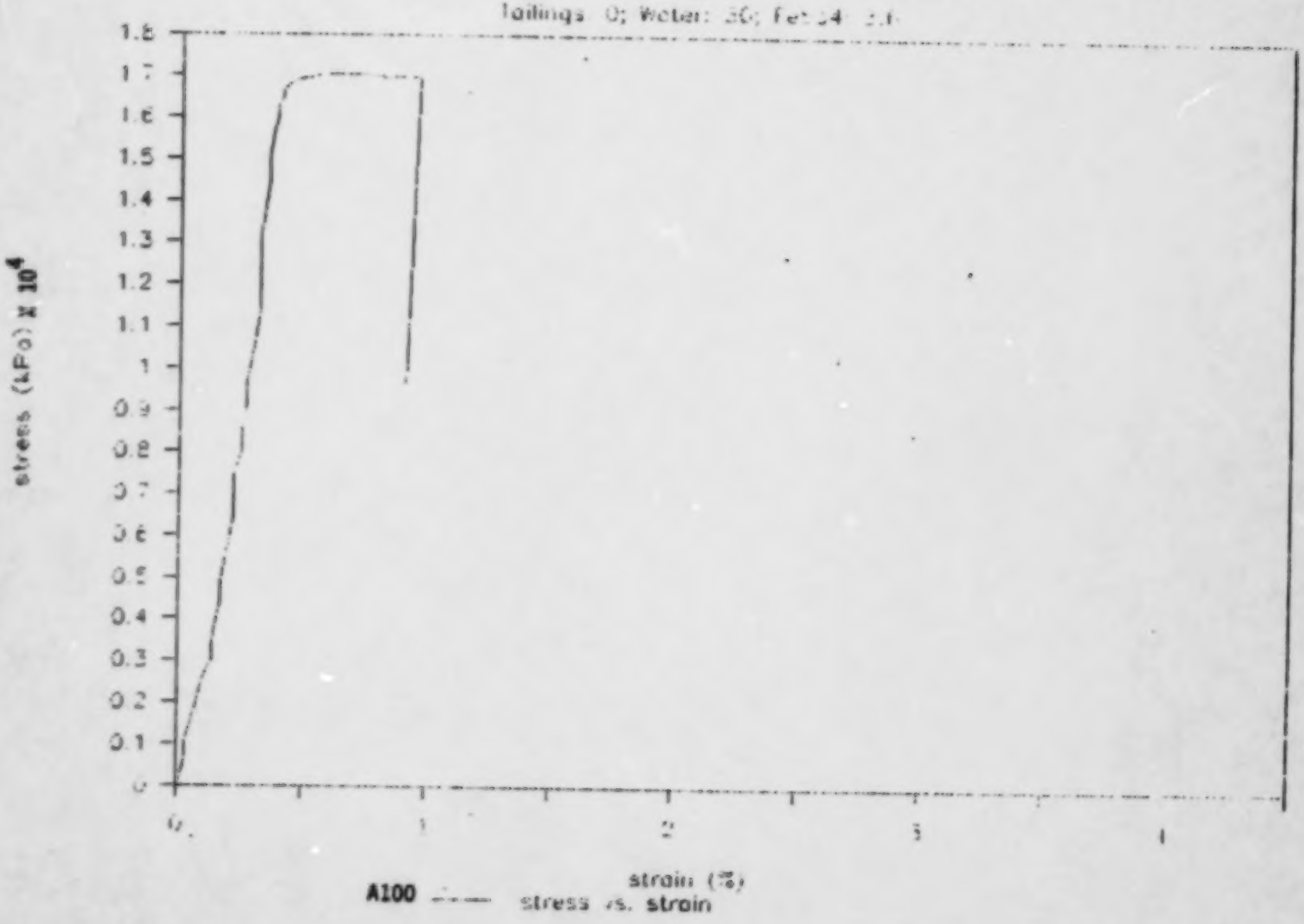

Anhydrite
Water 23; FeSO4: 5.6
stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A97
stress vs. strain

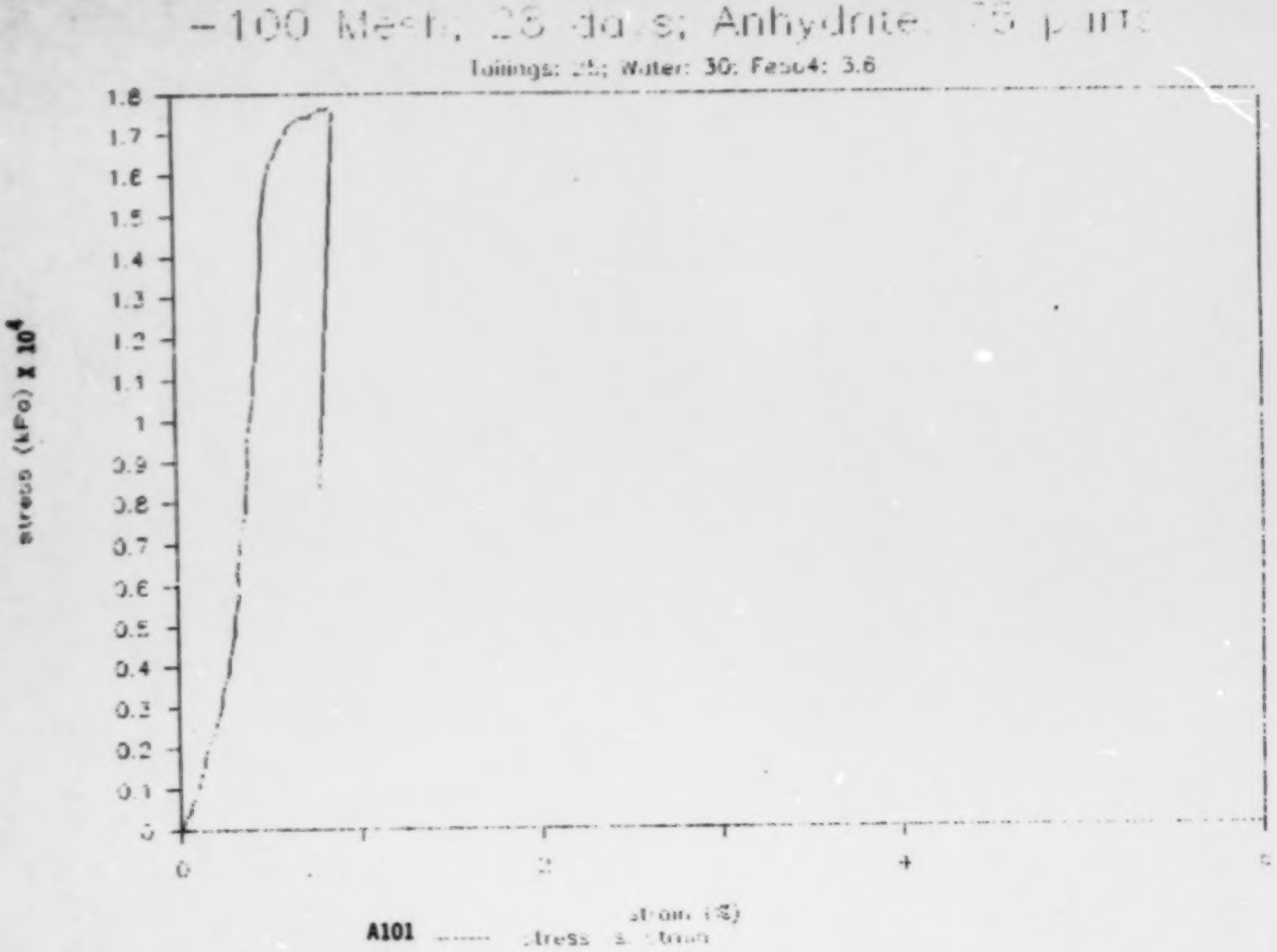


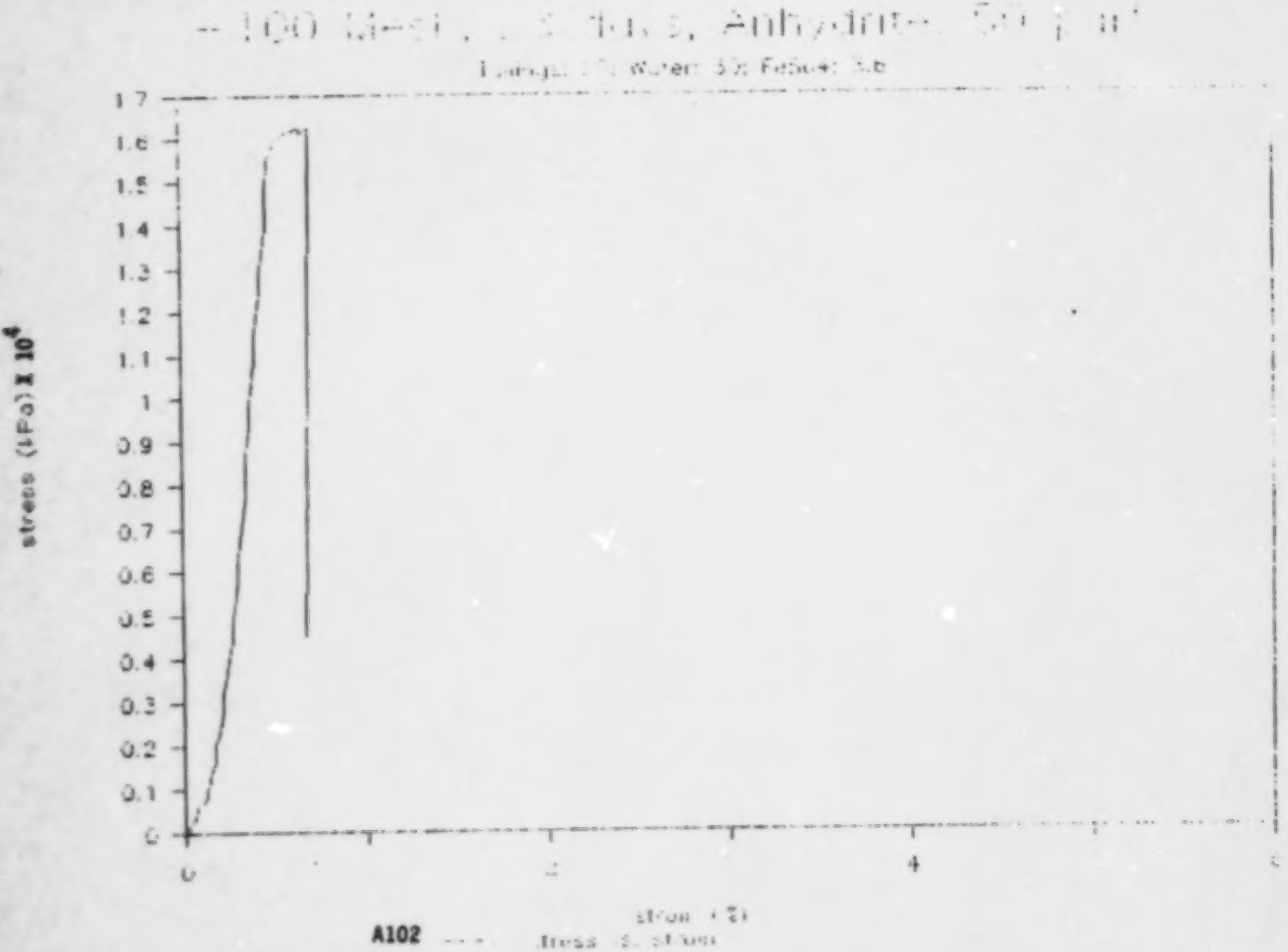
−100 Mesh; 28 days; Anhydrite: 50 parts

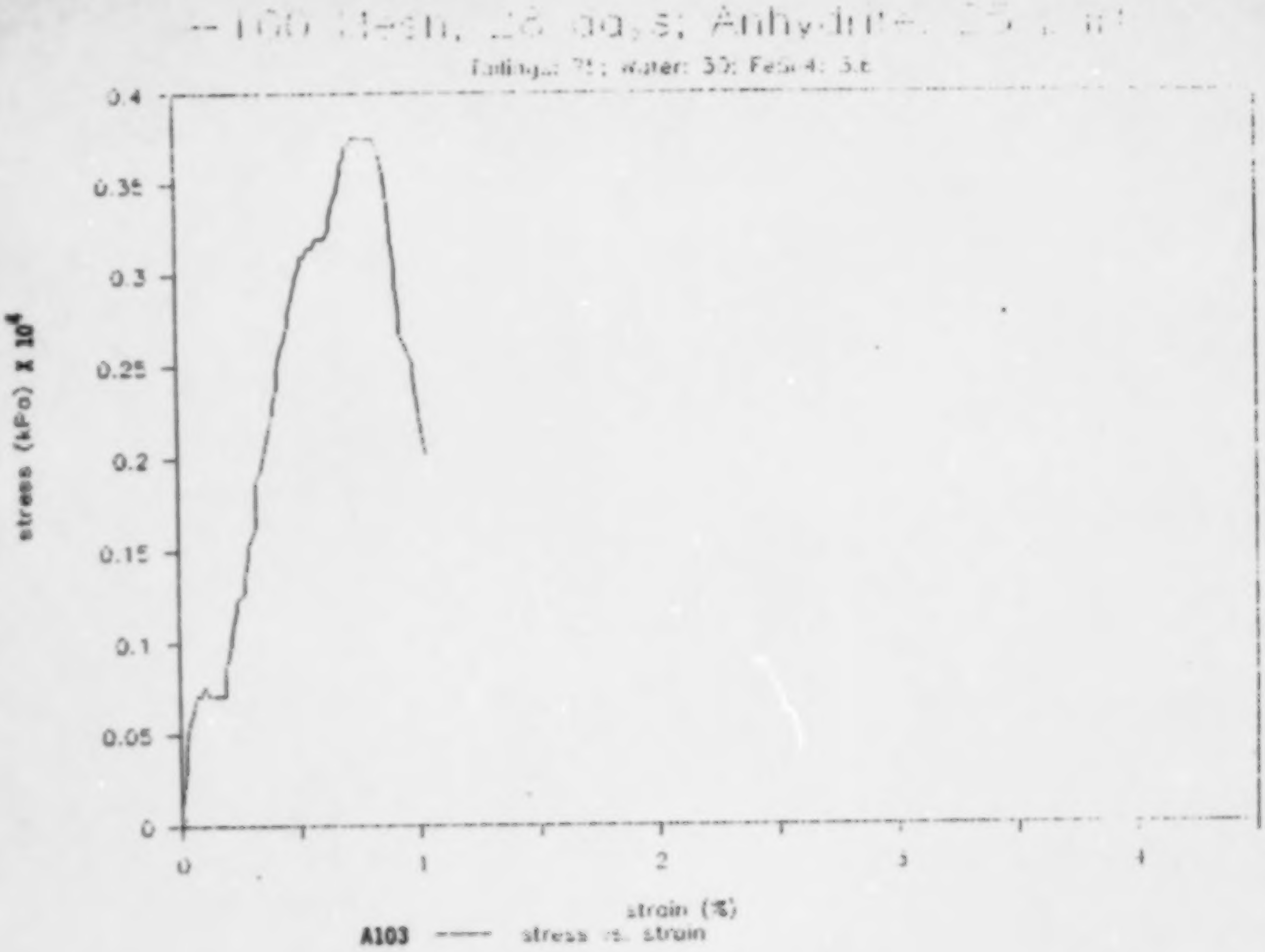
Tailings: 50; Water: 25; FeSo4: 3.6

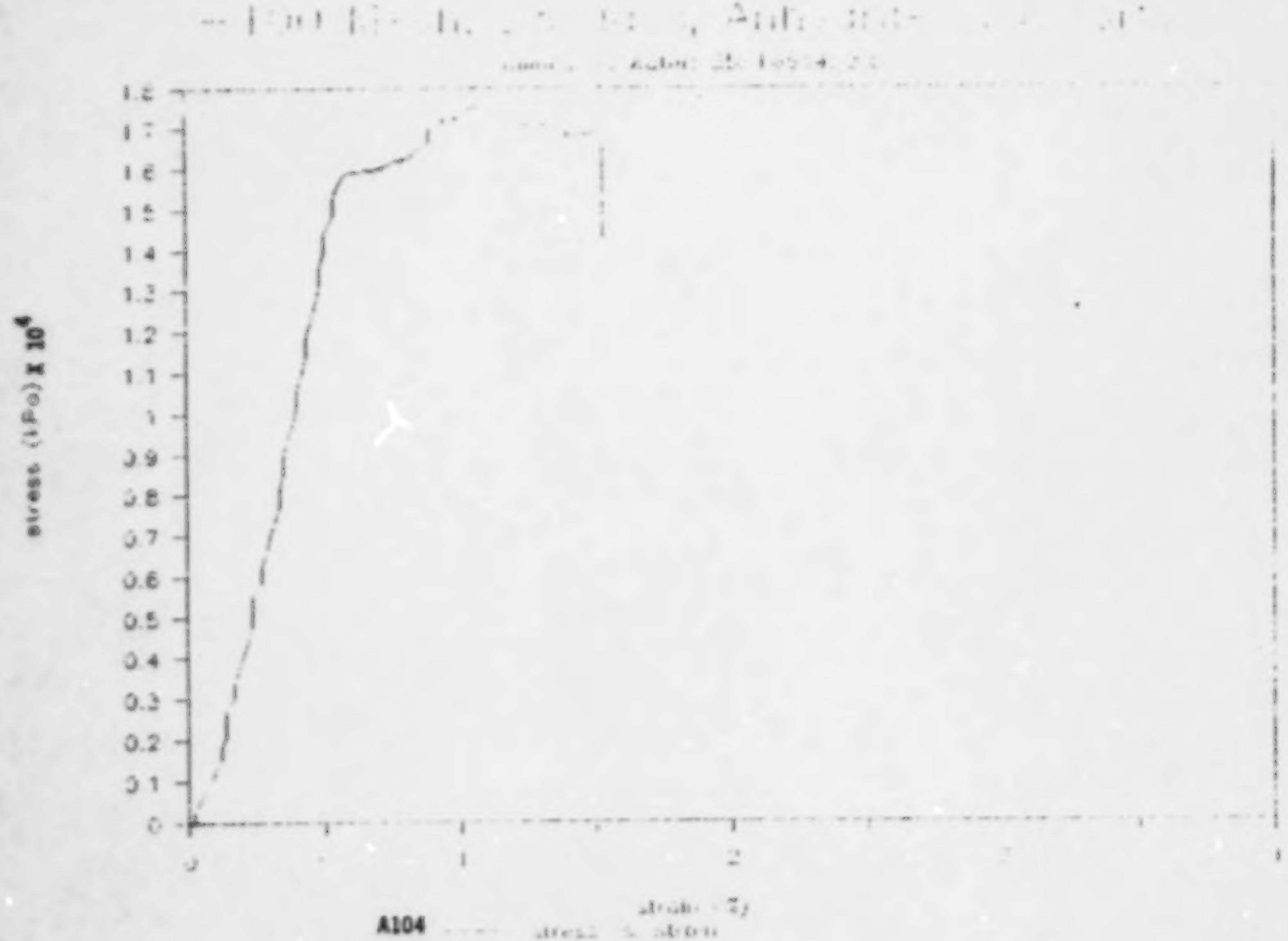

-100 Mesh; 28 days, Anhydrite: 25 parts
Tailings: 75; Water: 25; FeSo4: 3.0
stress (kPa) X 10^4
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A99 —— stress vs. strain

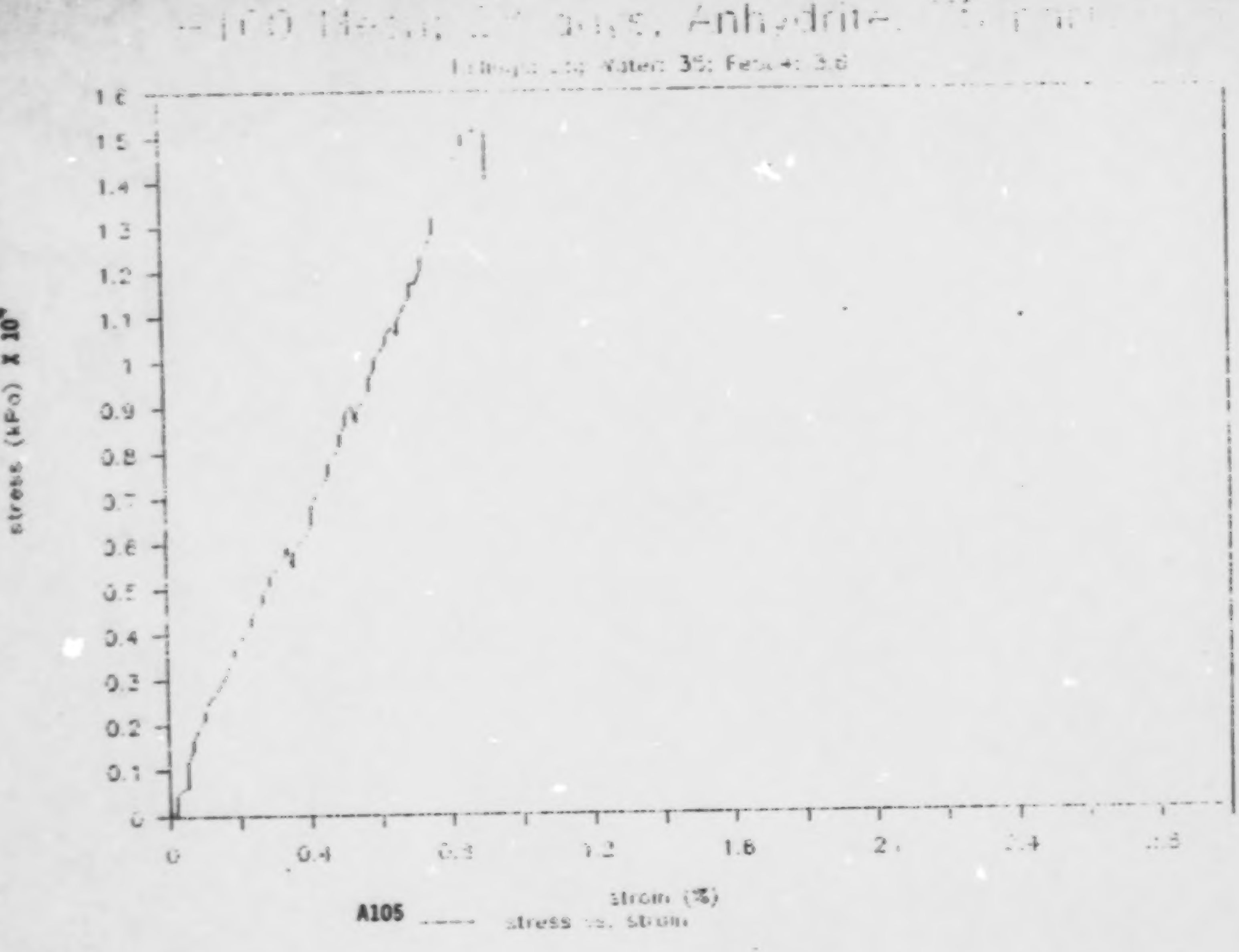

Tailings 0; Water: 30; Fe:34
stress (kPa) X 10^4
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A100
stress vs. strain

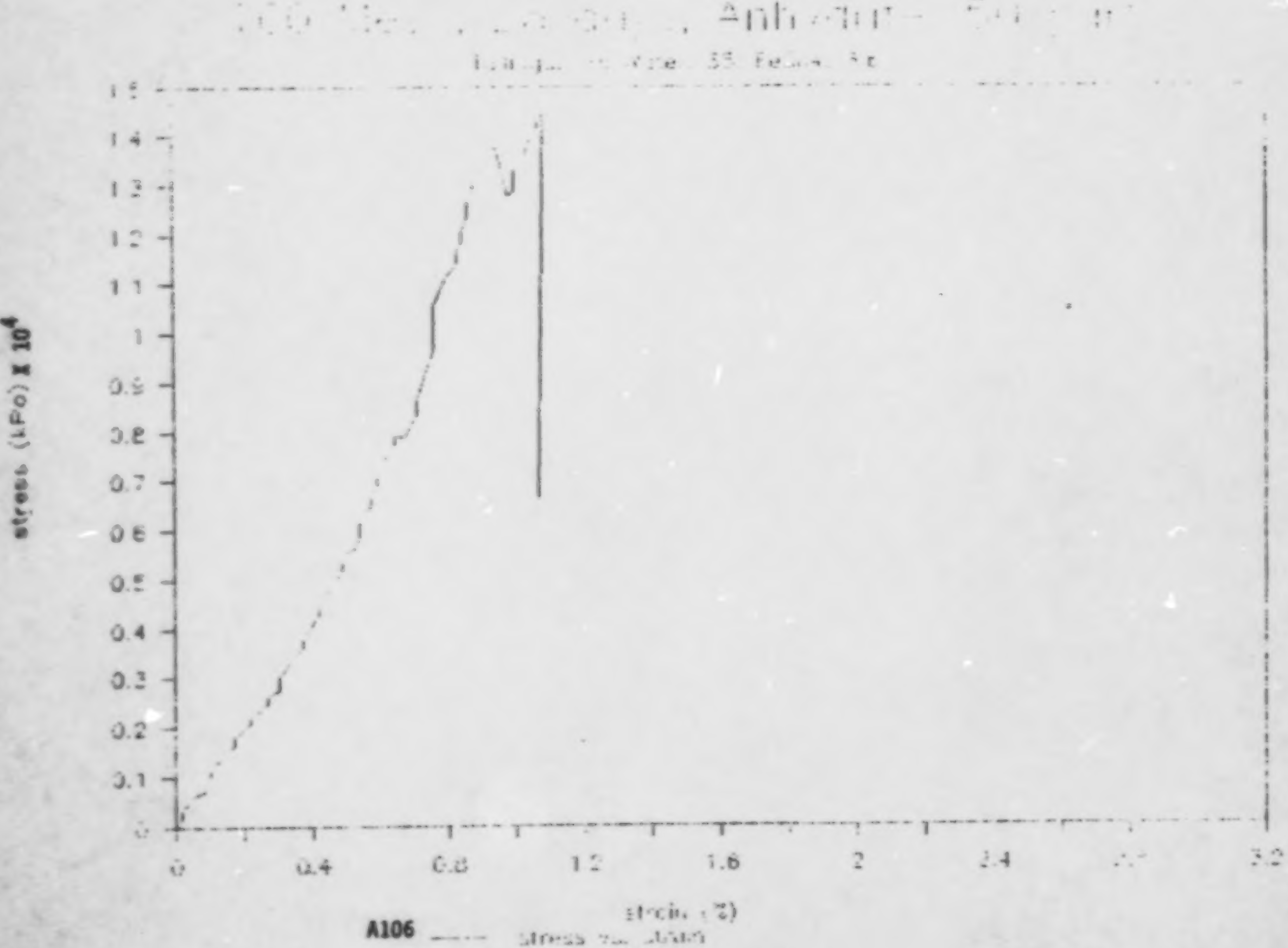

−100 Mesh, 28 days; Anhydrite 75 parts
Tailings: 25; Water: 30; FeSO4: 3.6
stress (kPa) X 10⁴
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
2
4
6
strain (%)
A101
stress vs. strain

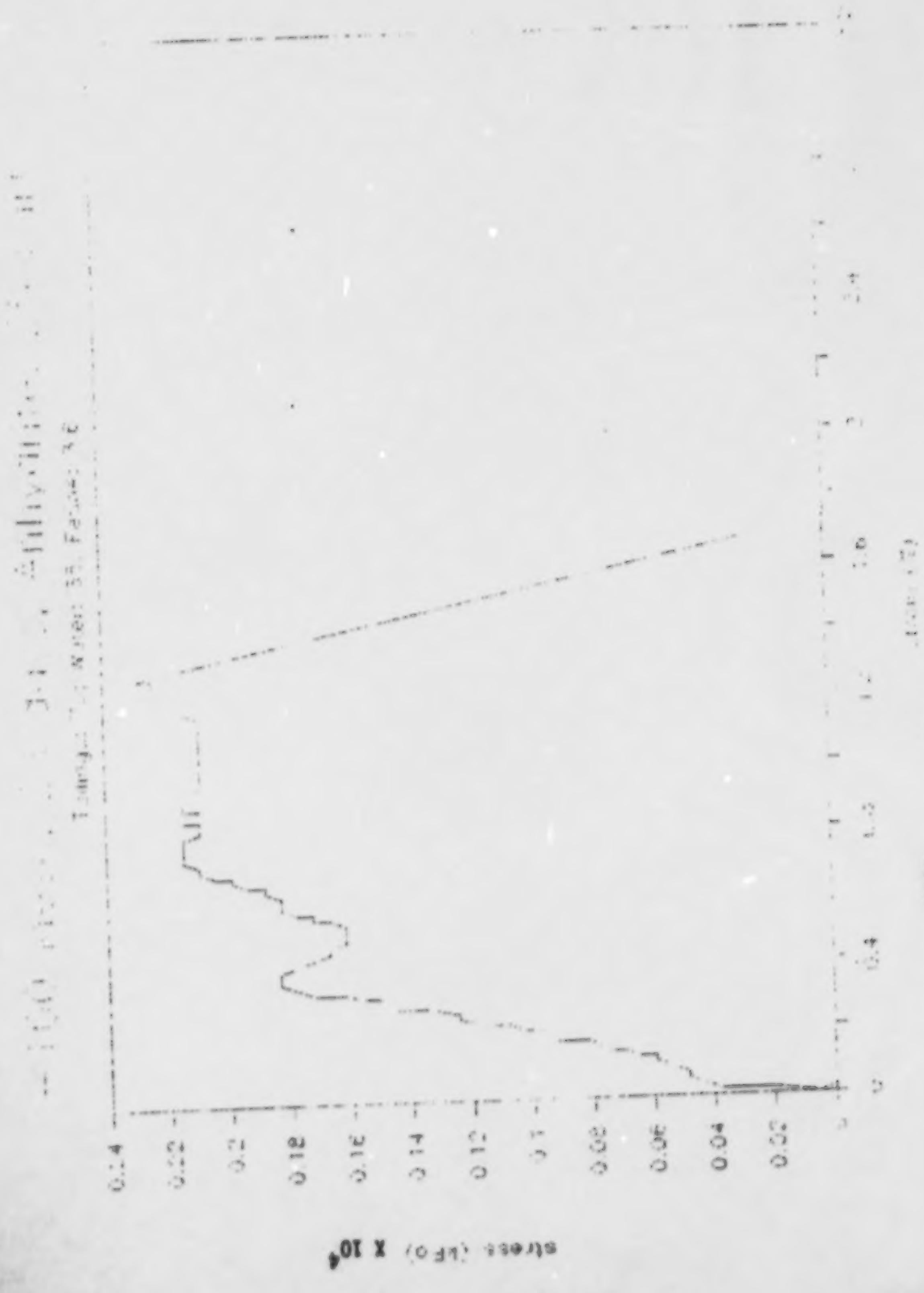

-100 Mesh, Anhydrite
Water 5% FeSO4 5.0
stress (kPa) X 10^4
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
2
4
6
strain (%)
A102
stress vs strain


-100 Mesh, 28 days; Anhydrite 25 [illegible]
Tailings: 71; water: 33; FeSO4: 3.6
stress (kPa) X 10⁴
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
3
4
strain (%)
A103 —— stress vs. strain


stress (1Pa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
0
1
2
4
strain (%)
A104
stress vs strain


Anhydrite
Water: 3%
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
stress (kPa) X 10^4
0
0.4
0.8
1.2
1.6
2
2.4
2.8
strain (%)
A105
stress vs. strain

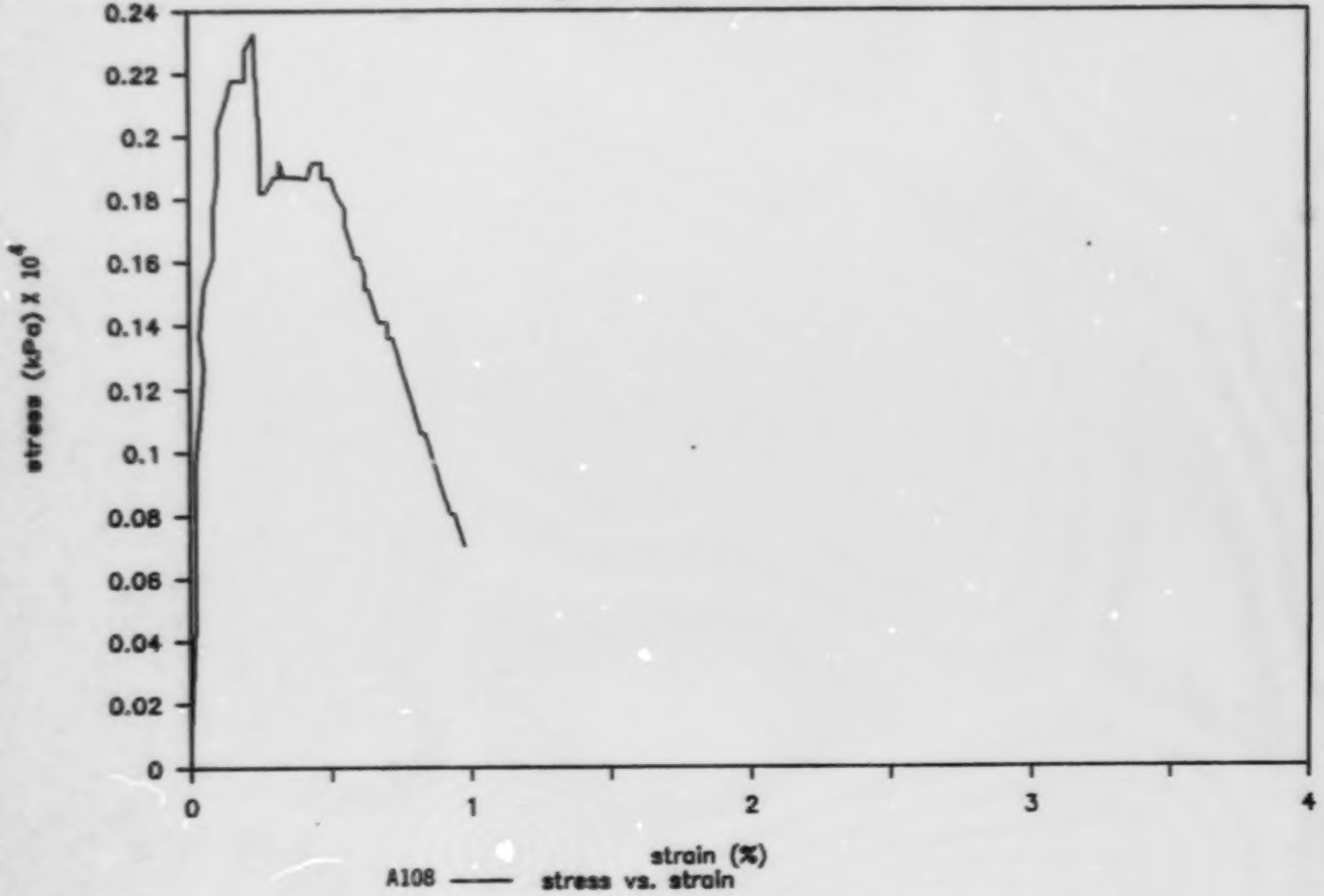

Anhydrite
stress (kPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
0
0.4
0.8
1.2
1.6
2
2.4
3.2
strain (%)
A106 —— stress vs. strain


Anhydrite
stress (MPa) X 10^4
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
0.4
0.8
1.2
1.6
2
2.4

A107

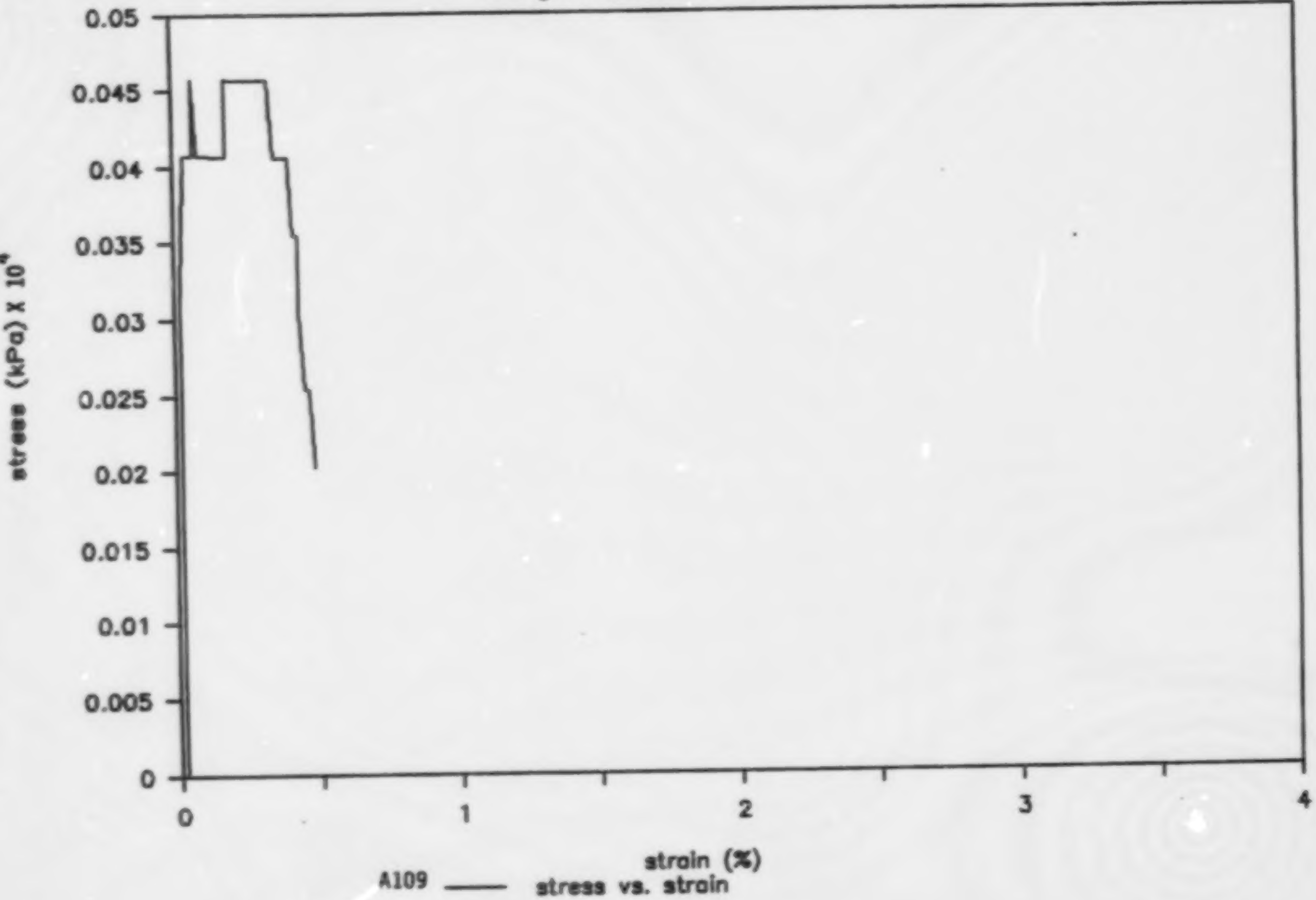
-50 Mesh; 7 days; Anhydrite: 50 parts

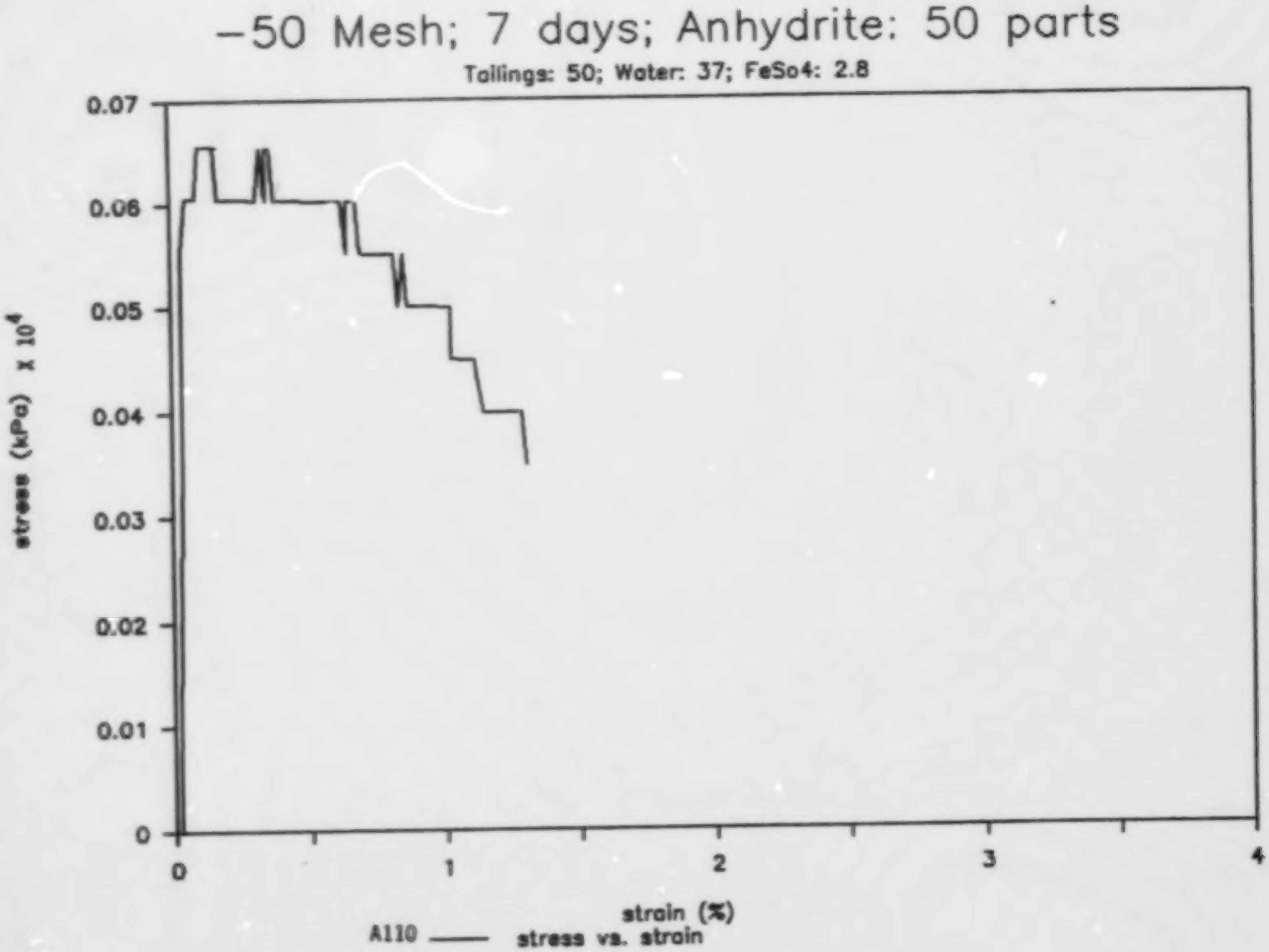
Tailings: 50; Water: 32; FeSo4: 2.8



−50 Mesh; 7 days; Anhydrite: 25 parts

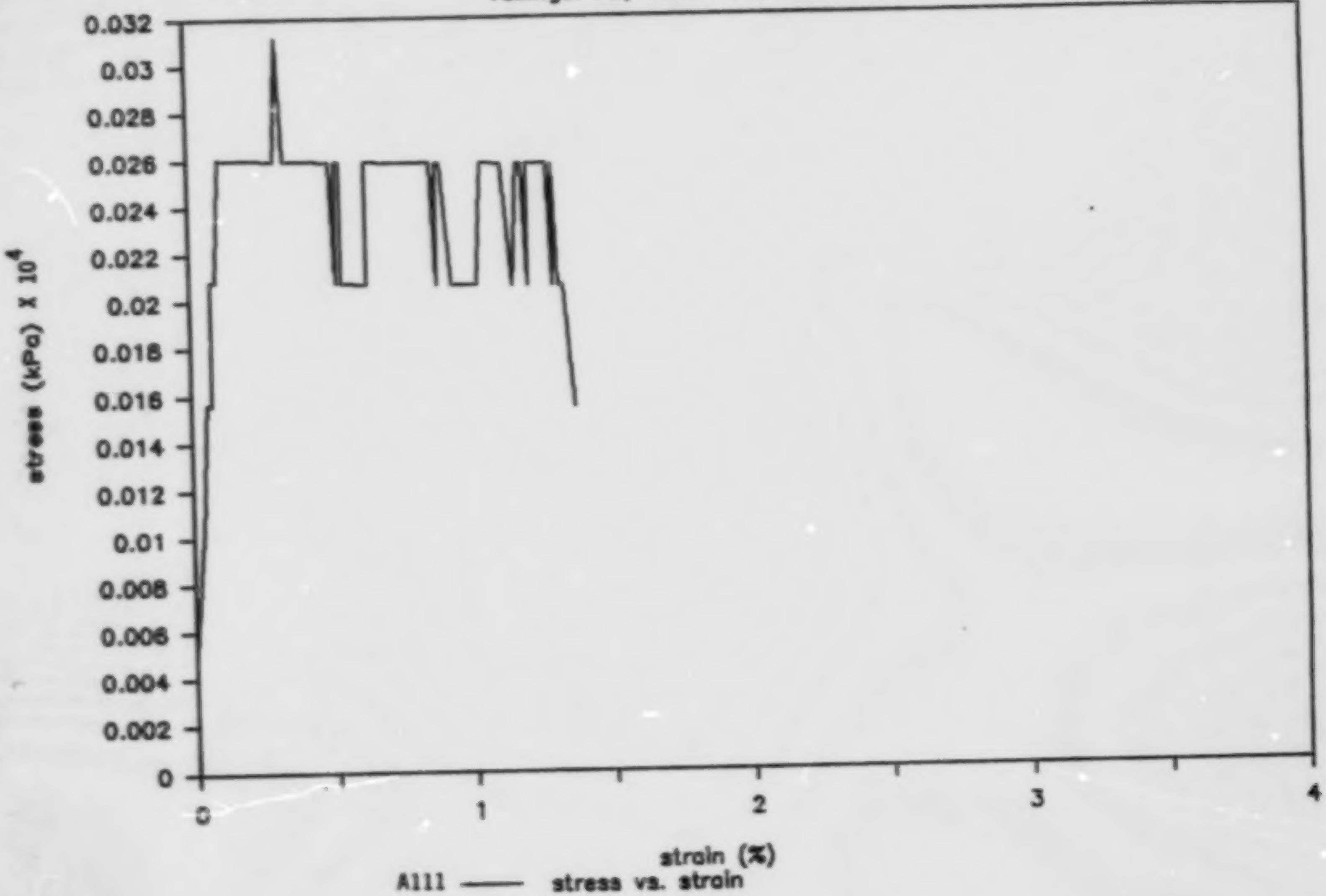
Tailings: 75; Water: 32; FeSo4: 2.8

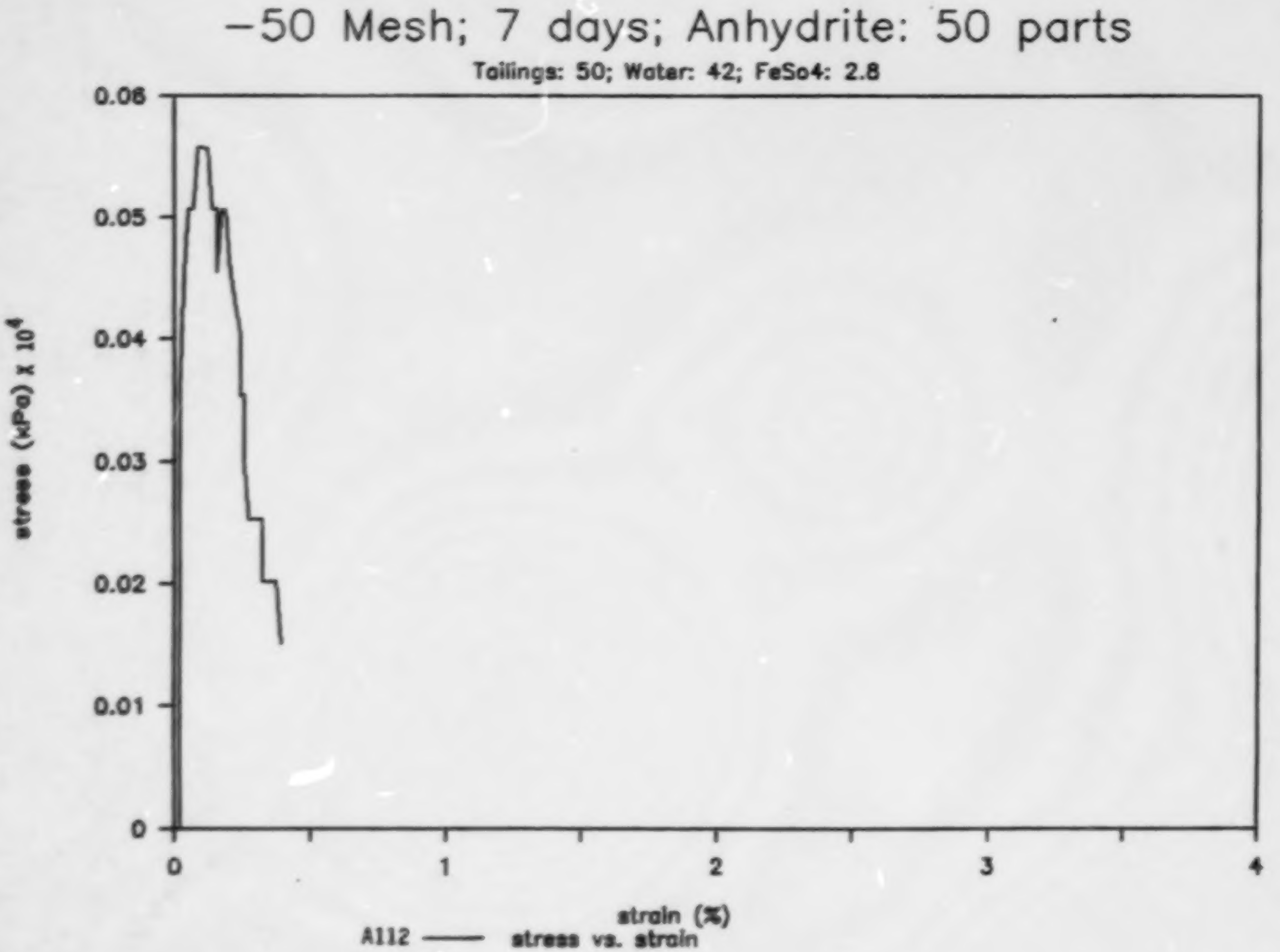



−50 Mesh; 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 37; FeSo4: 2.8
stress (kPa) x 10^4
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A110 —— stress vs. strain

−50 Mesh; 7 days; Anhydrite: 25 parts

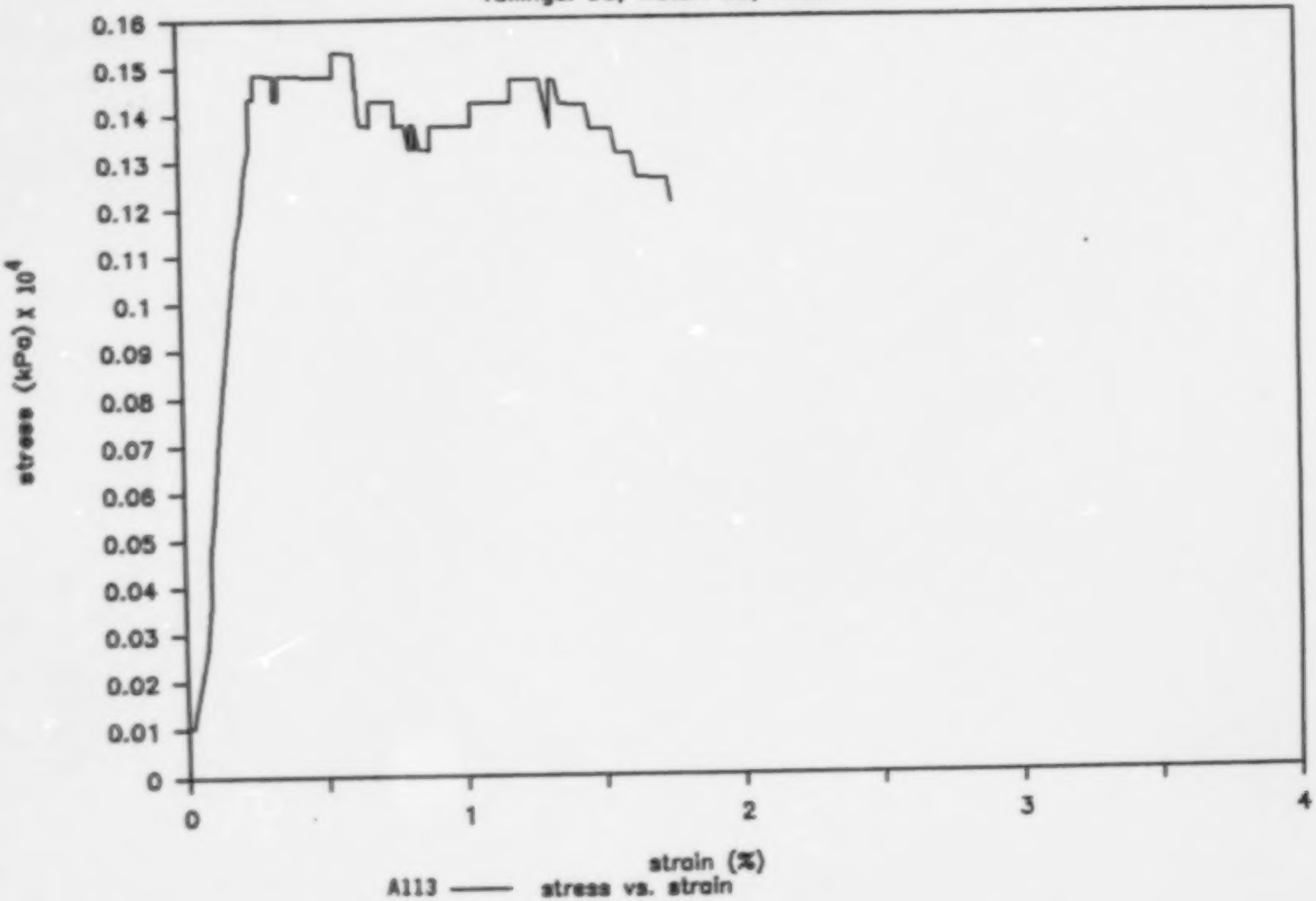
Tailings: 75; Water: 37; FeSo4: 2.8

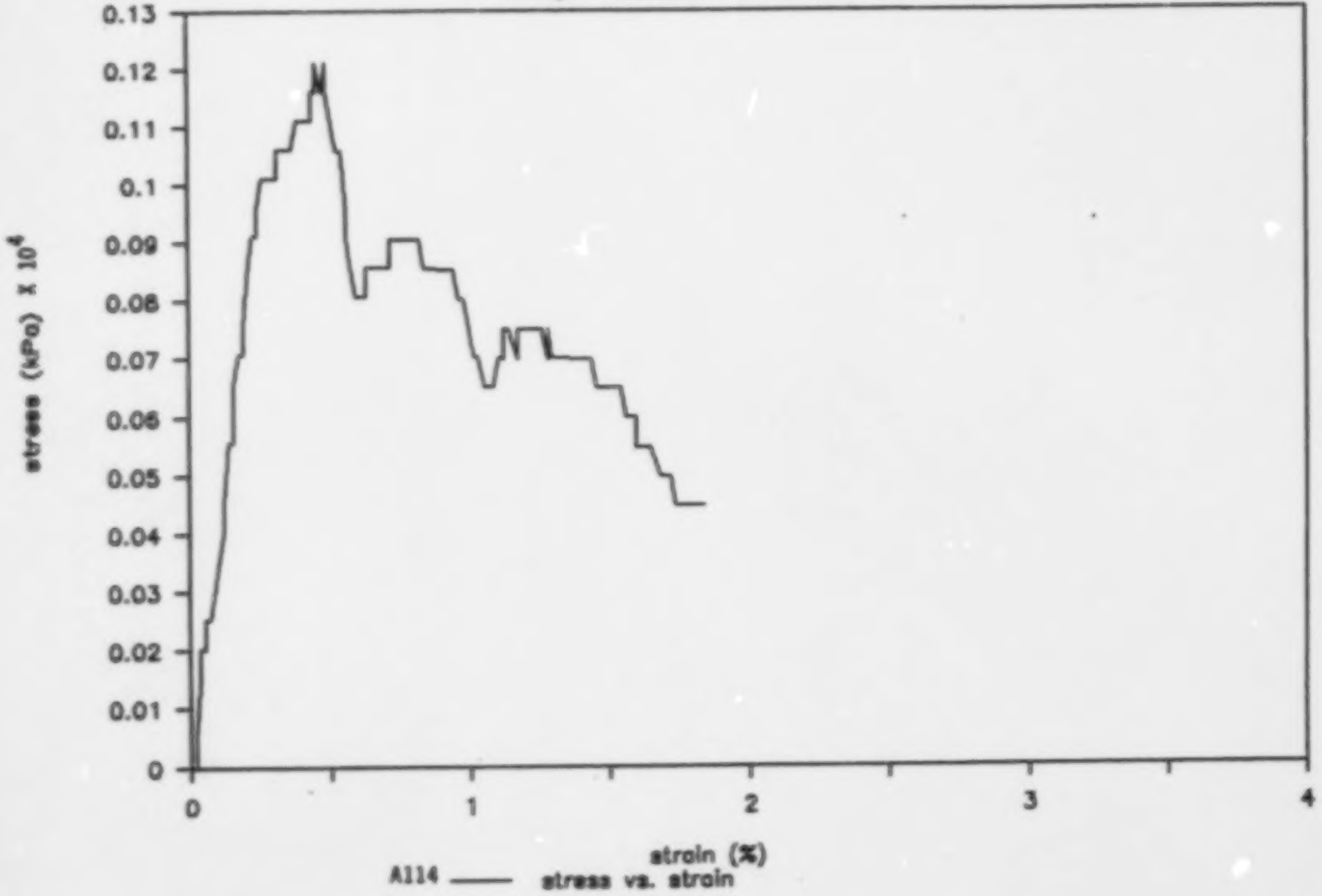



-50 Mesh; 7 days; Anhydrite: 50 parts
Tailings: 50; Water: 42; FeSo4: 2.8
stress (kPa) x 10^4
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A112 —— stress vs. strain

-50 Mesh; 7 days; Anhydrite: 50 parts

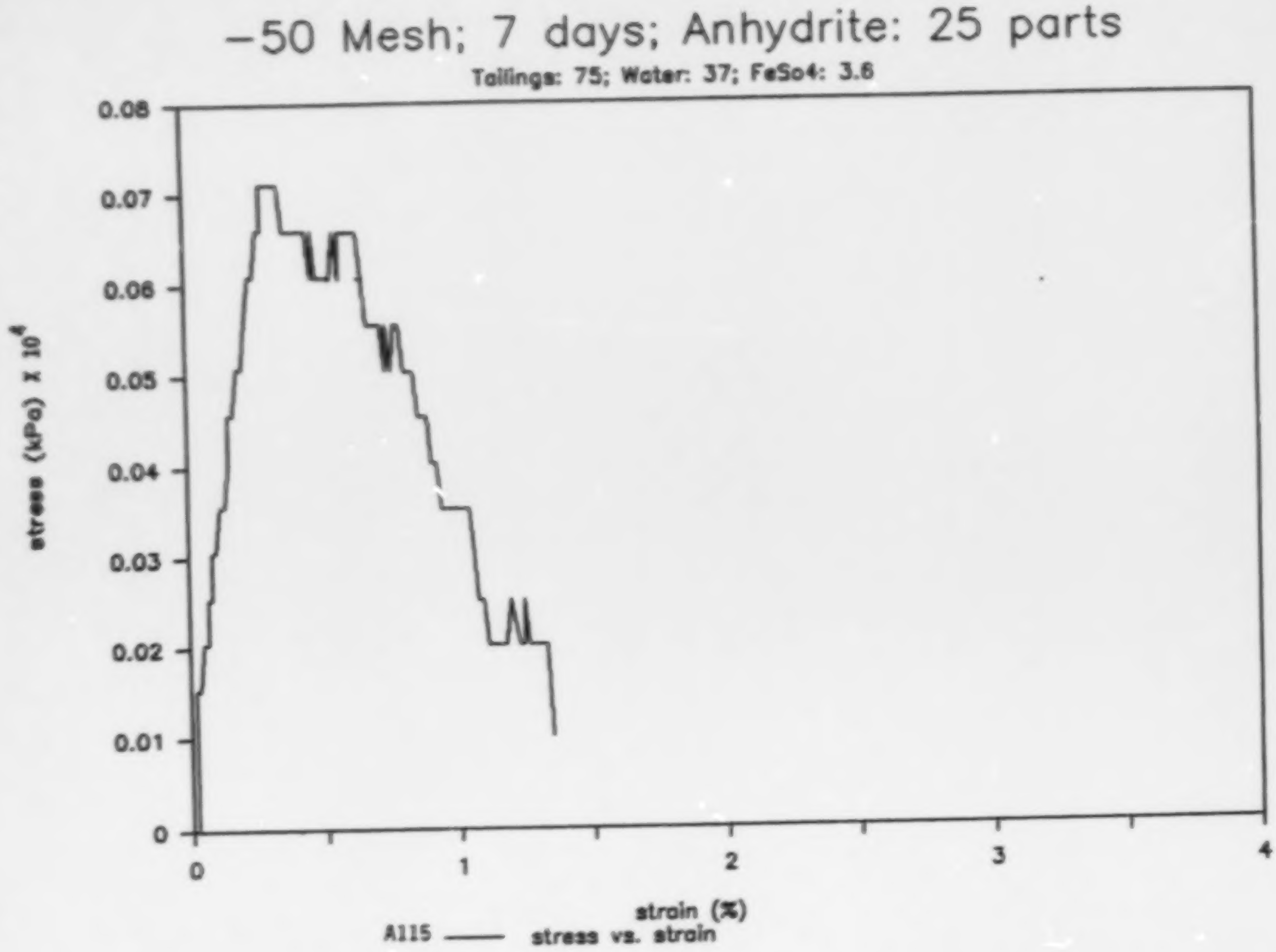
Tailings: 50; Water: 32; FeSo4: 3.6





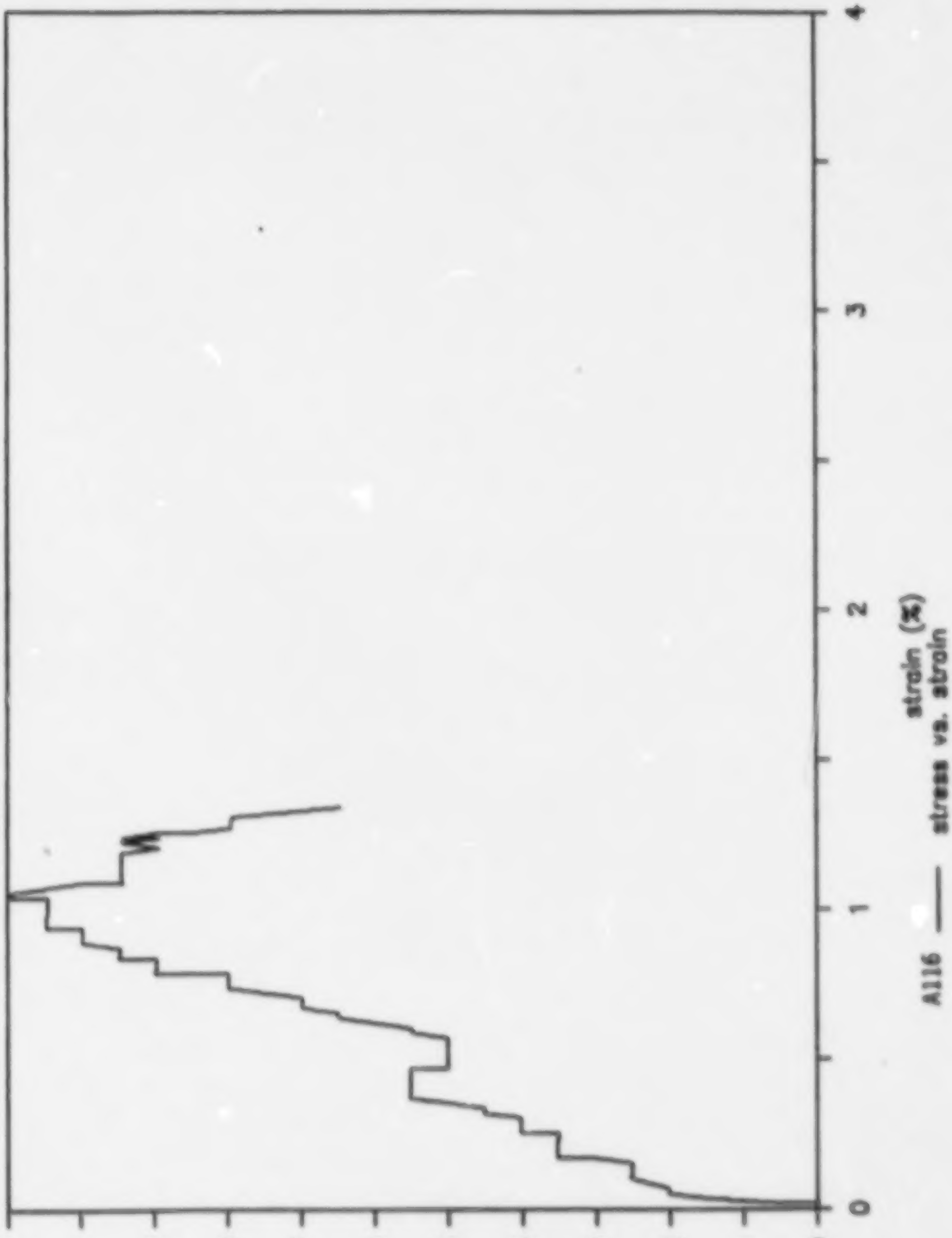
−50 Mesh; 7 days; Anhydrite: 50 parts

Tailings: 50; Water: 37; FeSo4: 3.6


−50 Mesh; 7 days; Anhydrite: 25 parts
Tailings: 75; Water: 37; FeSo4: 3.6
stress (kPa) X 10⁴
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A115 —— stress vs. strain

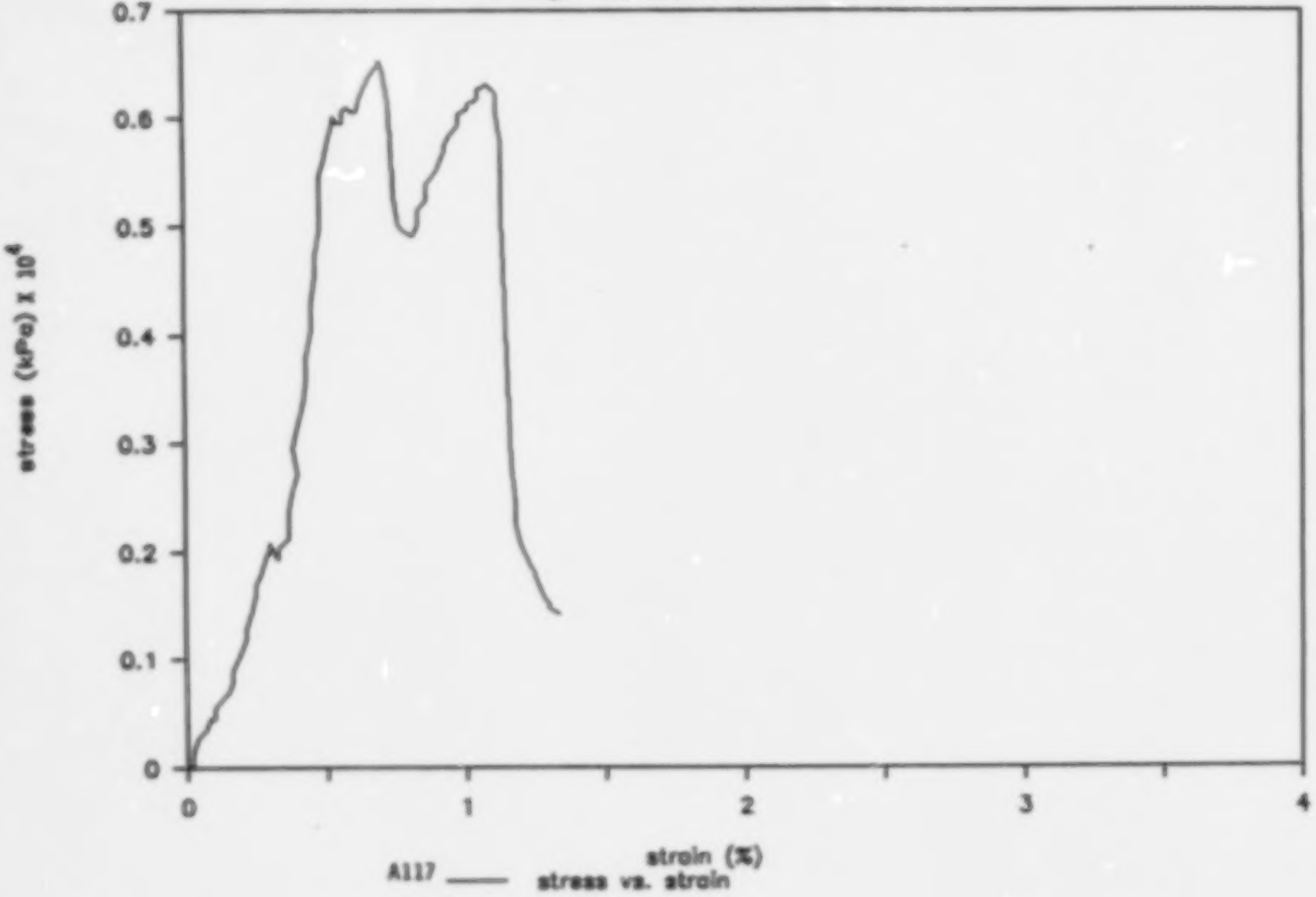
-50 Mesh; 7 days; Anhydrite: 50 parts

Tailings: 50; Water: 42; FeSo4: 3.6



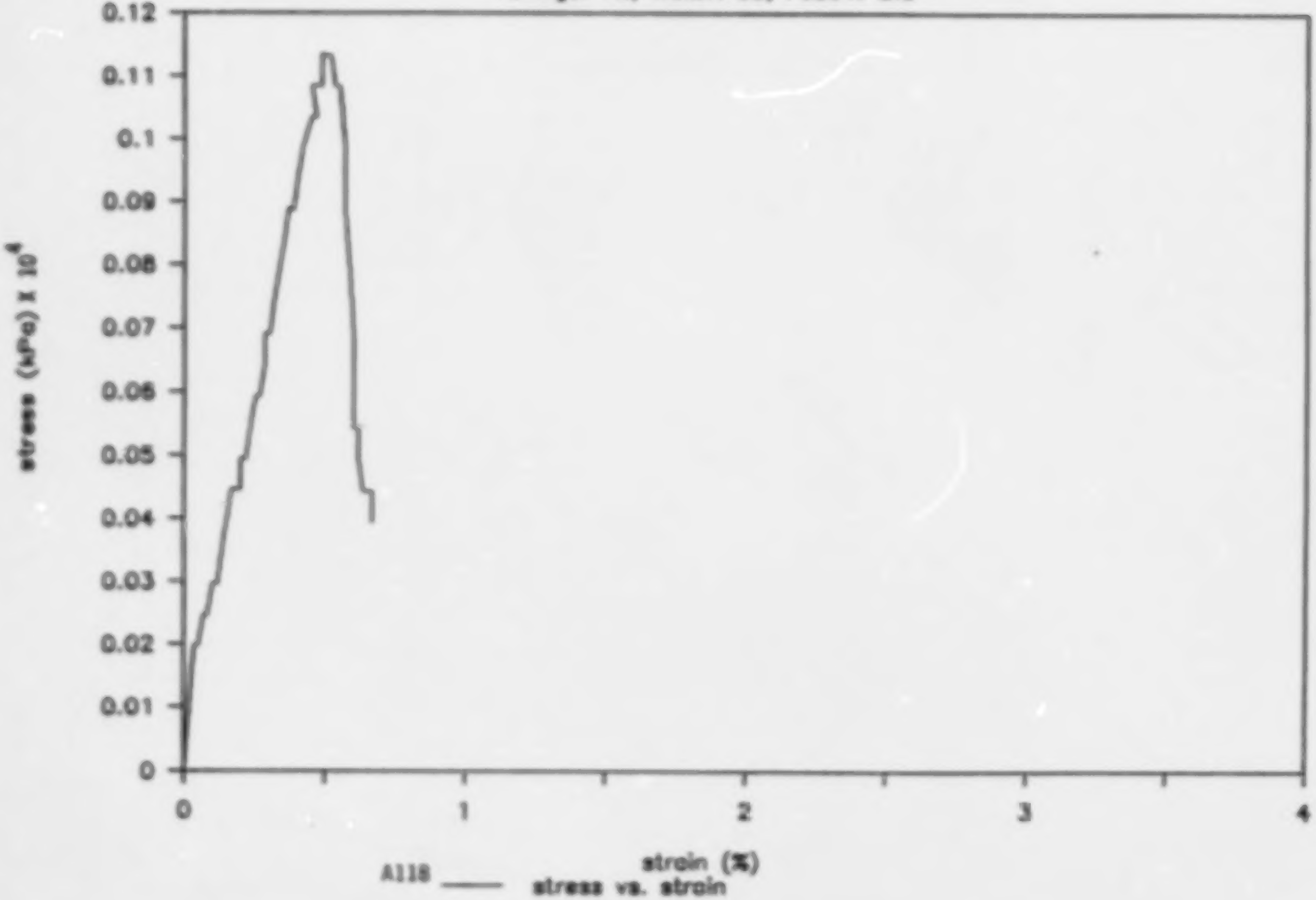
−50 Mesh; 14 days; Anhydrite: 50 parts

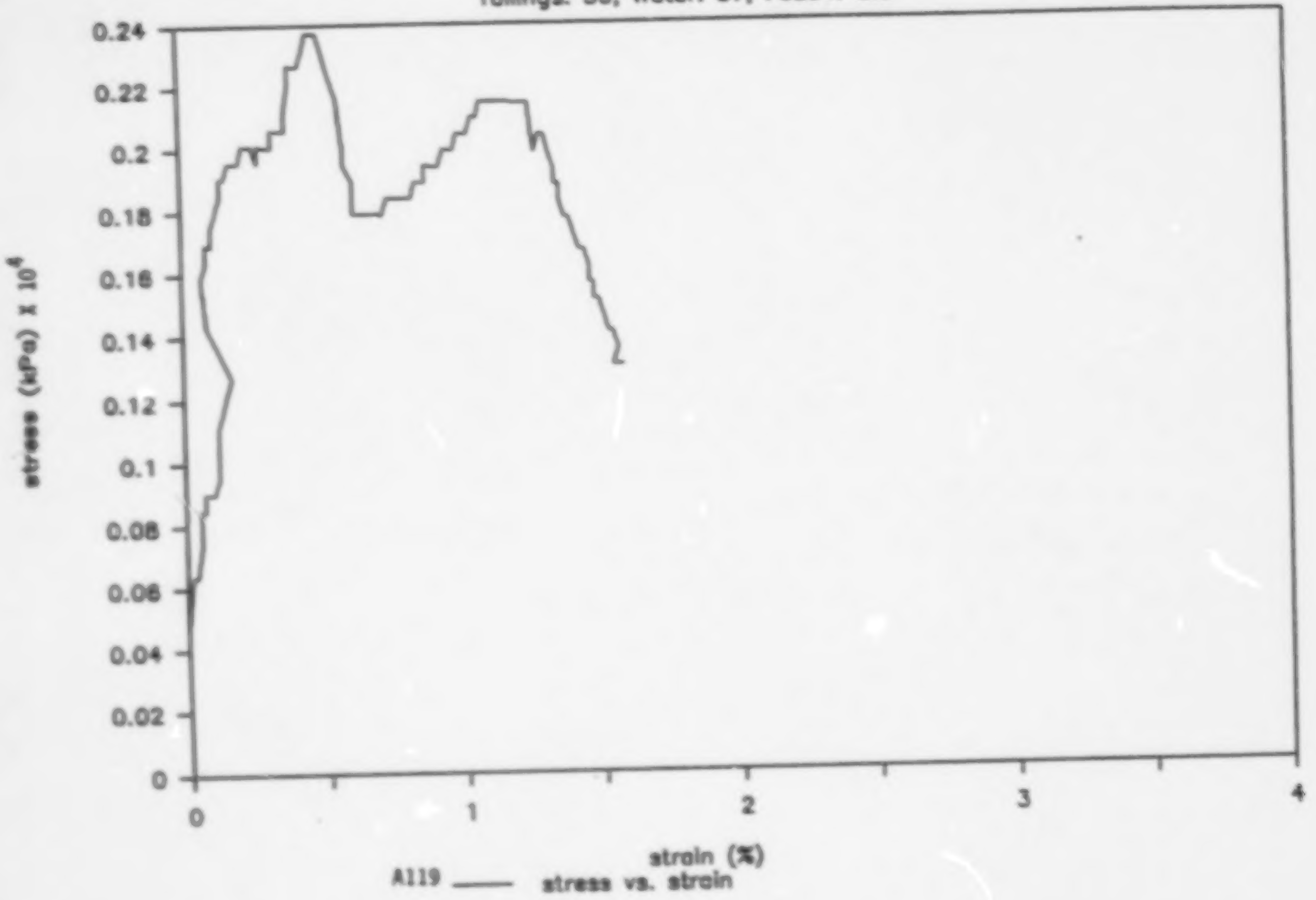
Tailings: 50; Water: 32; FeSo4: 2.8



−50 Mesh; 14 days; Anhydrite: 25 parts

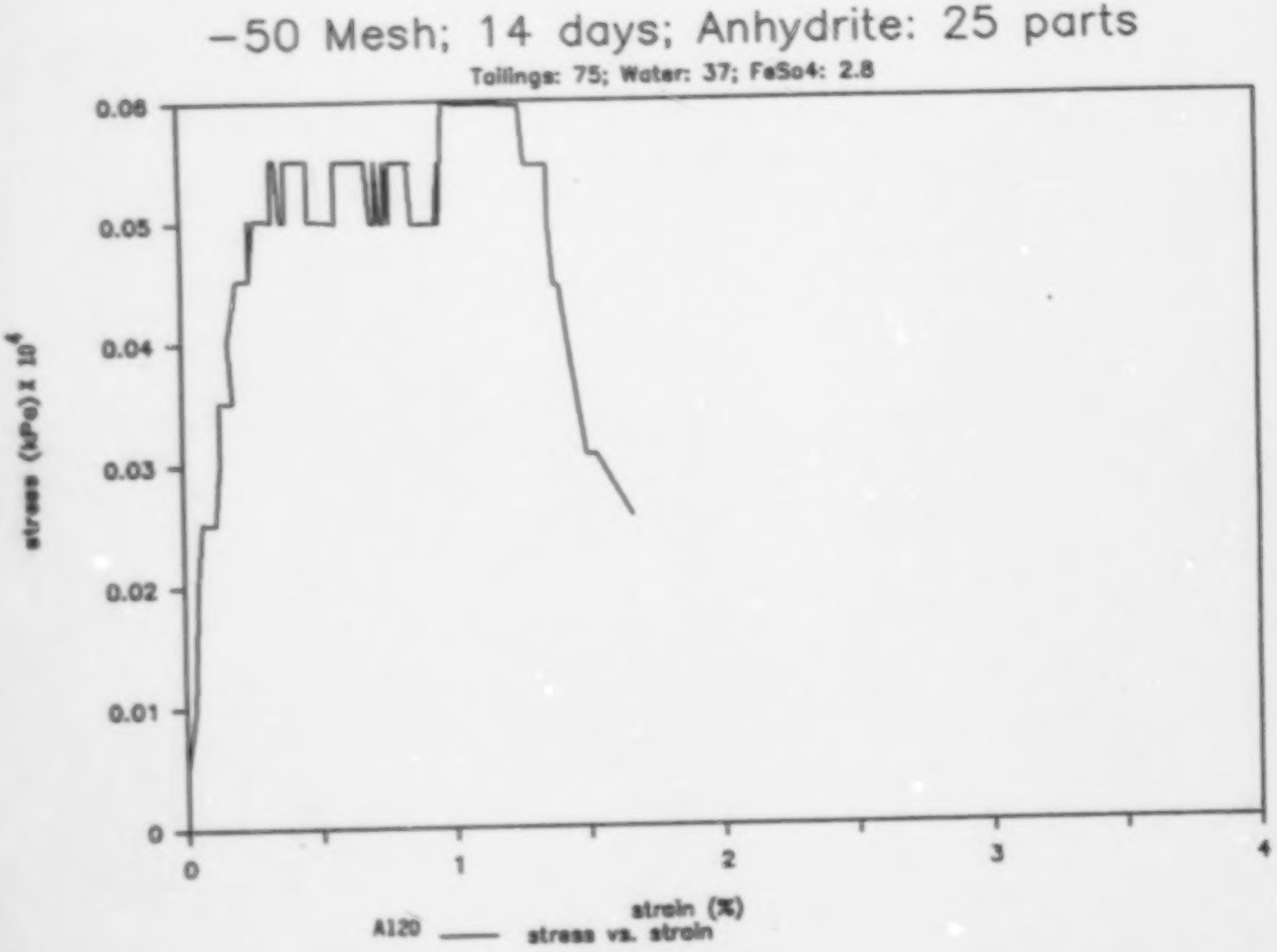
Tailings: 75; Water: 32; FeSo4: 2.8





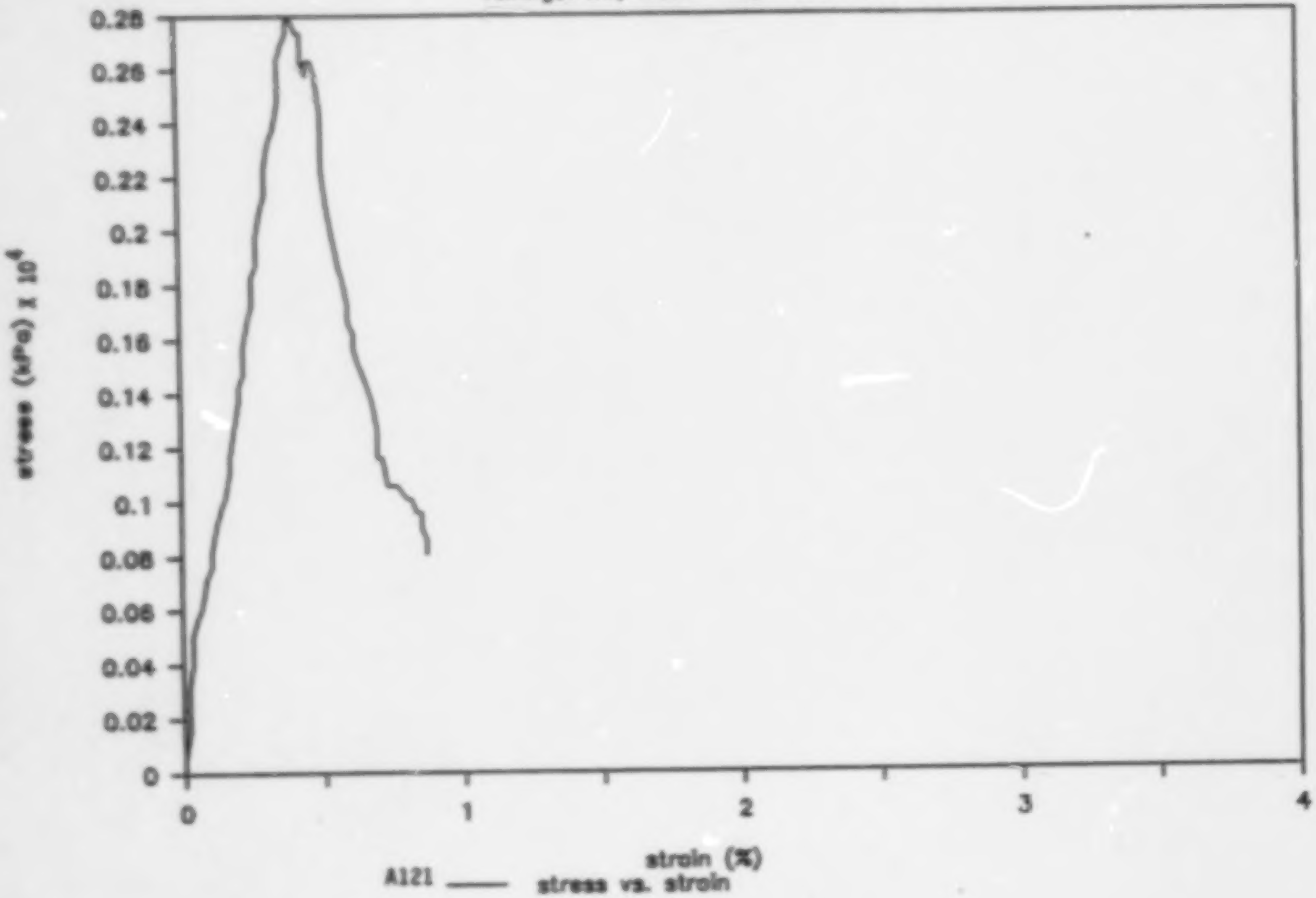
−50 Mesh; 14 days; Anhydrite: 50 parts

Tailings: 50; Water: 37; FeSo4: 2.8


−50 Mesh; 14 days; Anhydrite: 25 parts
Tailings: 75; Water: 37; FeSo4: 2.8
stress (kPa) X 10^4
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A120 —— stress vs. strain

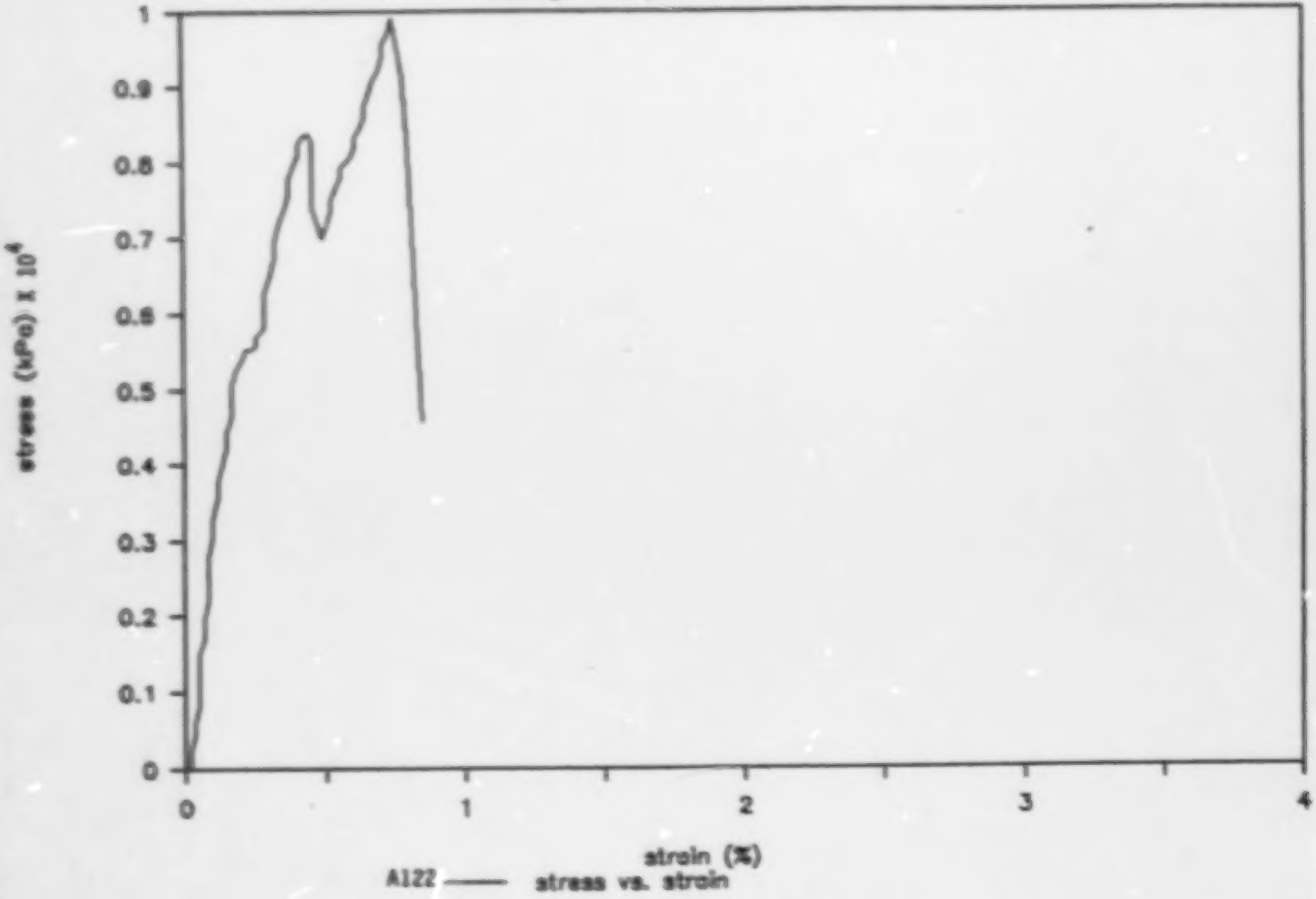
−50 Mesh; 14 days; Anhydrite: 50 parts

Tailings: 50; Water: 42; FeSo4: 2.8



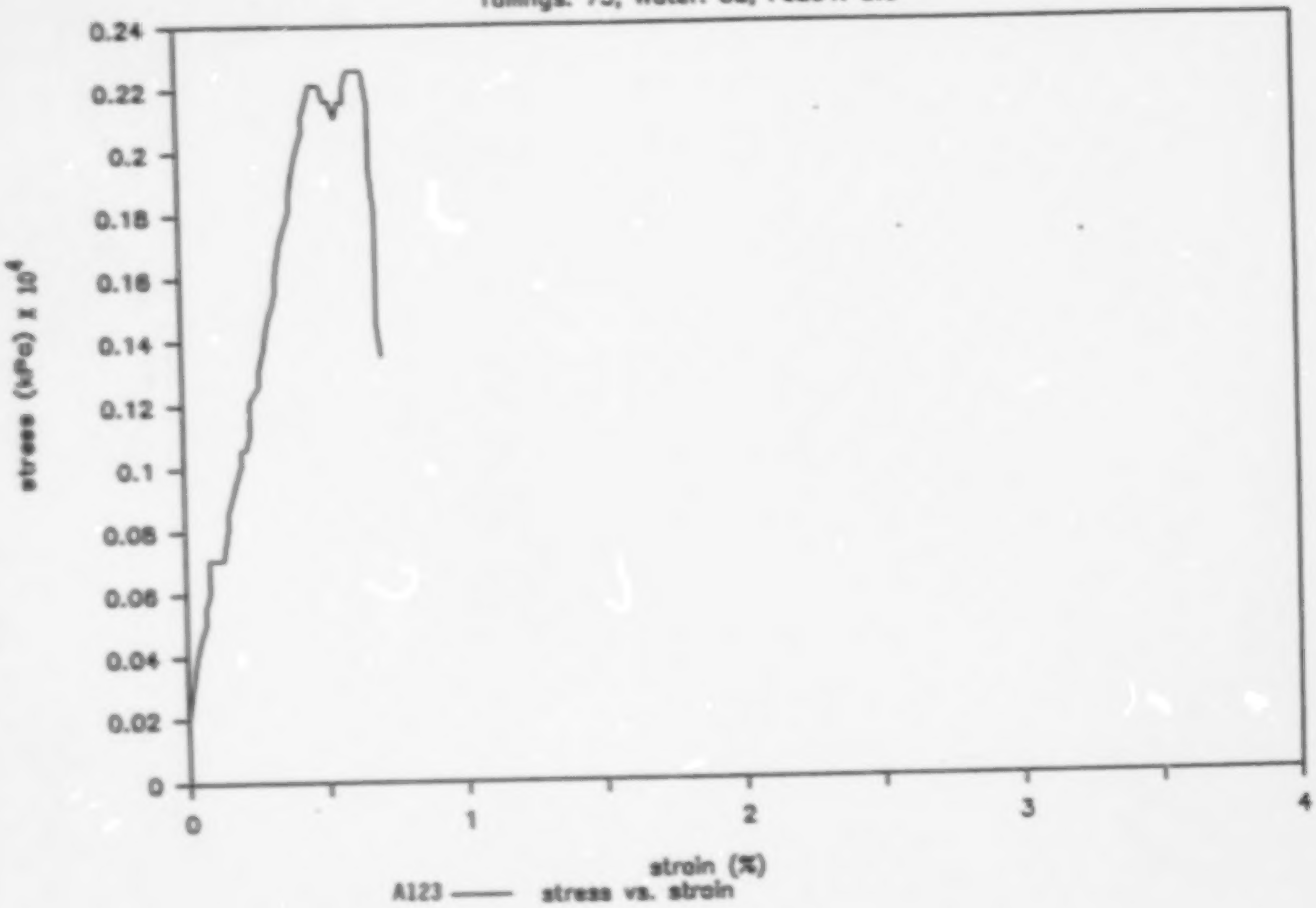
-50 Mesh; 14 days; Anhydrite: 50 parts

Tailings: 50; Water: 32; FeSo4: 3.6



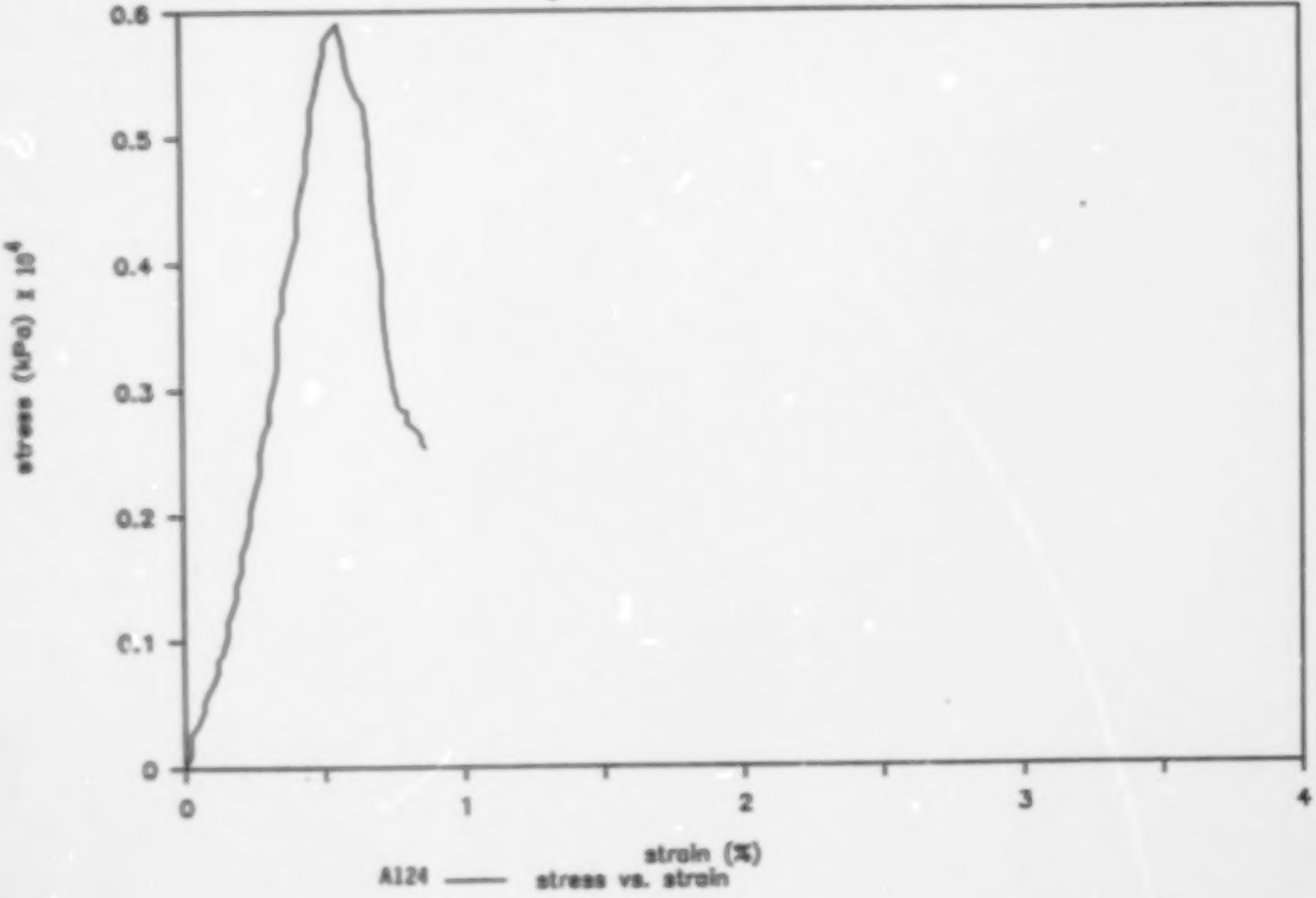
−50 Mesh; 14 days; Anhydrite: 25 parts

Tailings: 75; Water: 32; FeSo4: 3.6



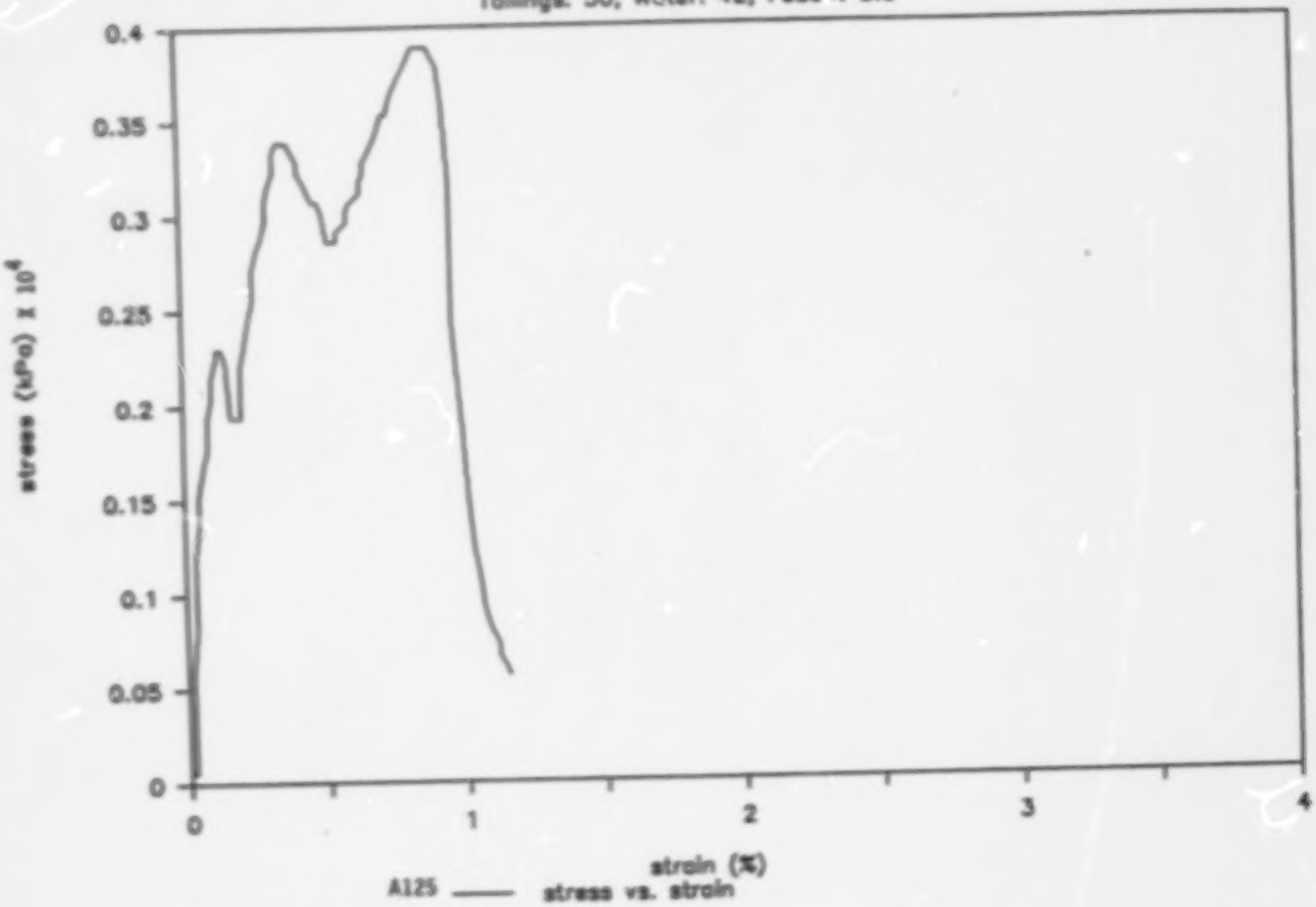
−50 Mesh; 14 days; Anhydrite: 50 parts

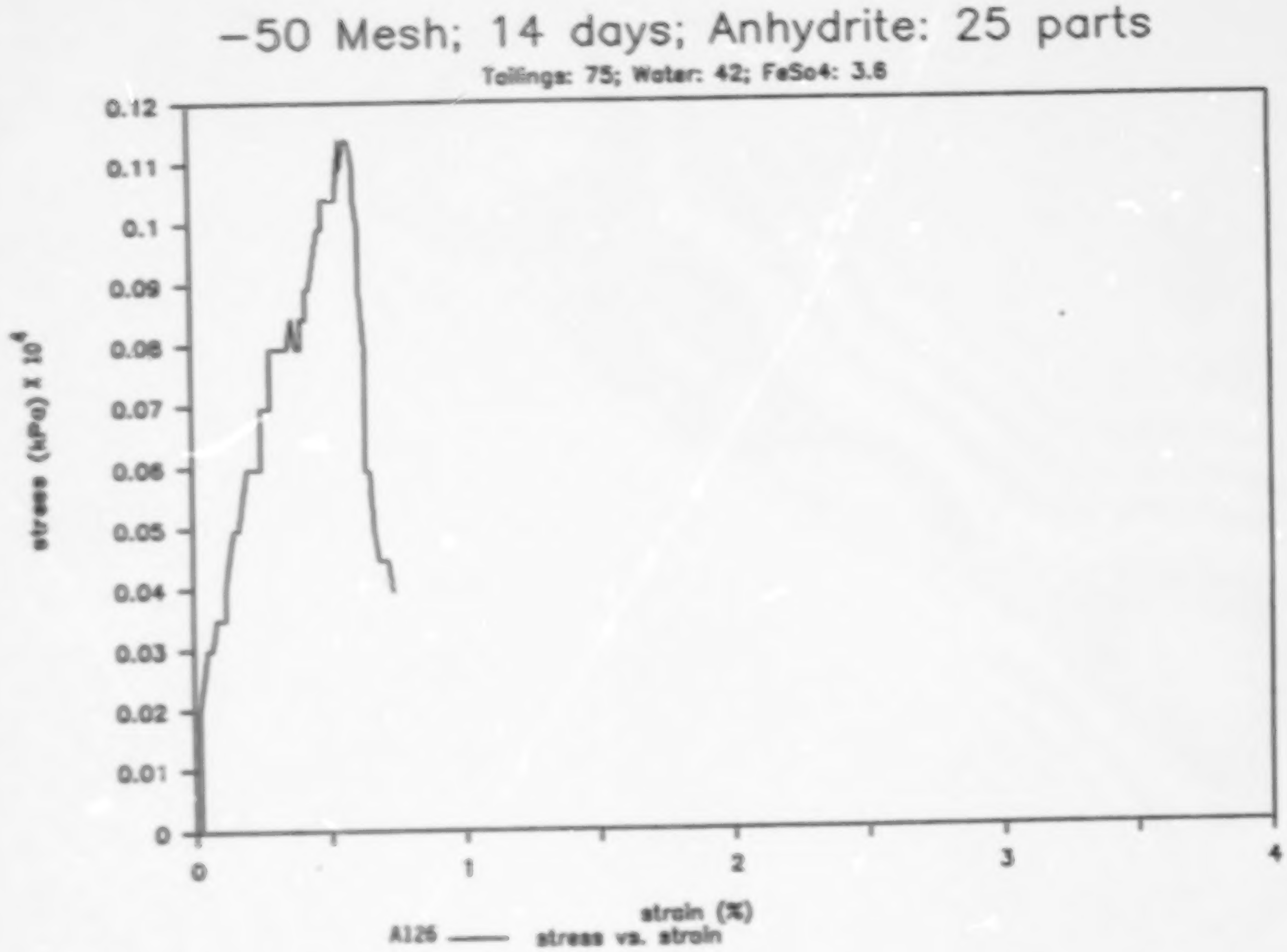
Tailings: 50; Water: 37; FeSo4: 3.6

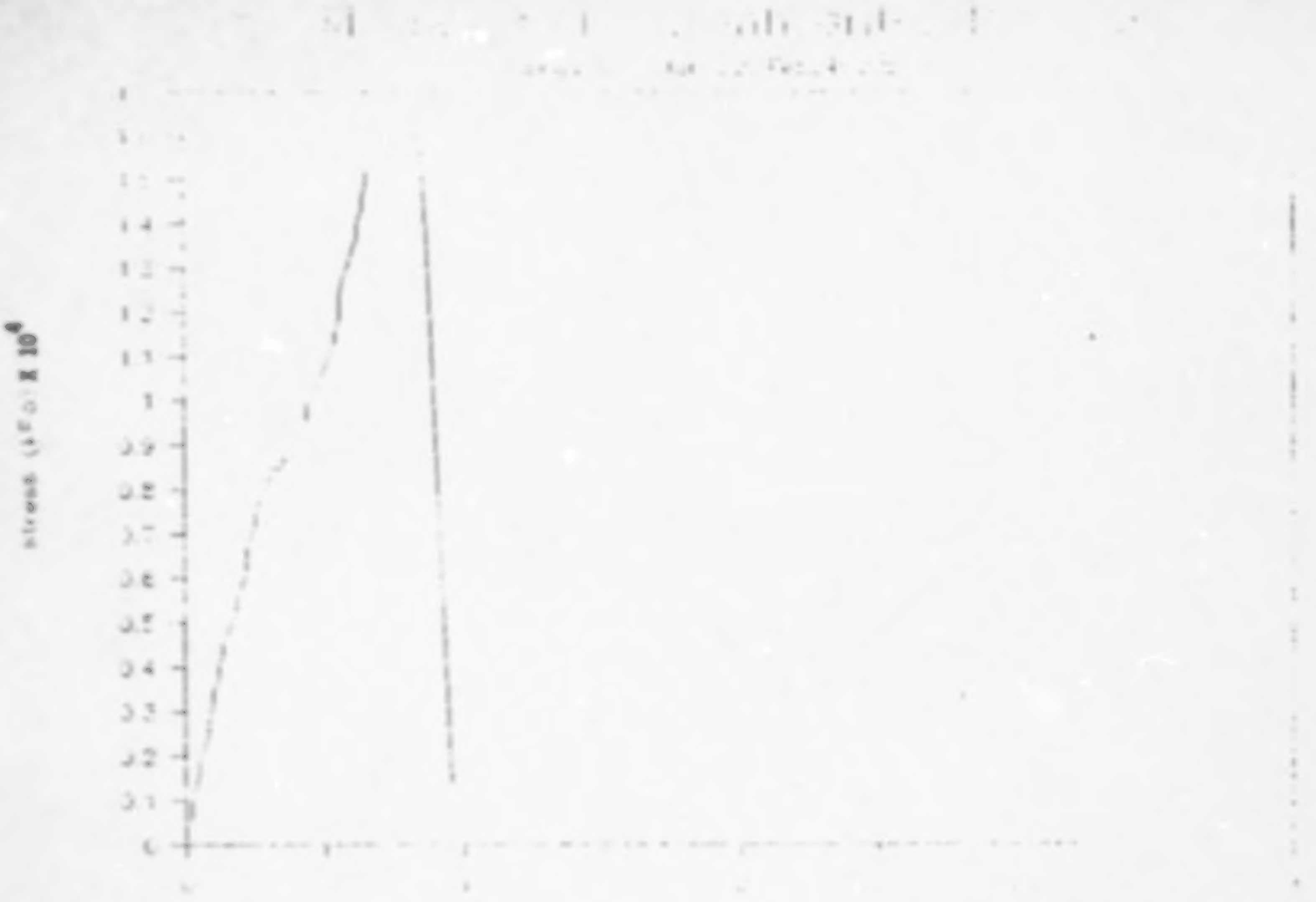


−50 Mesh; 14 days; Anhydrite: 50 parts

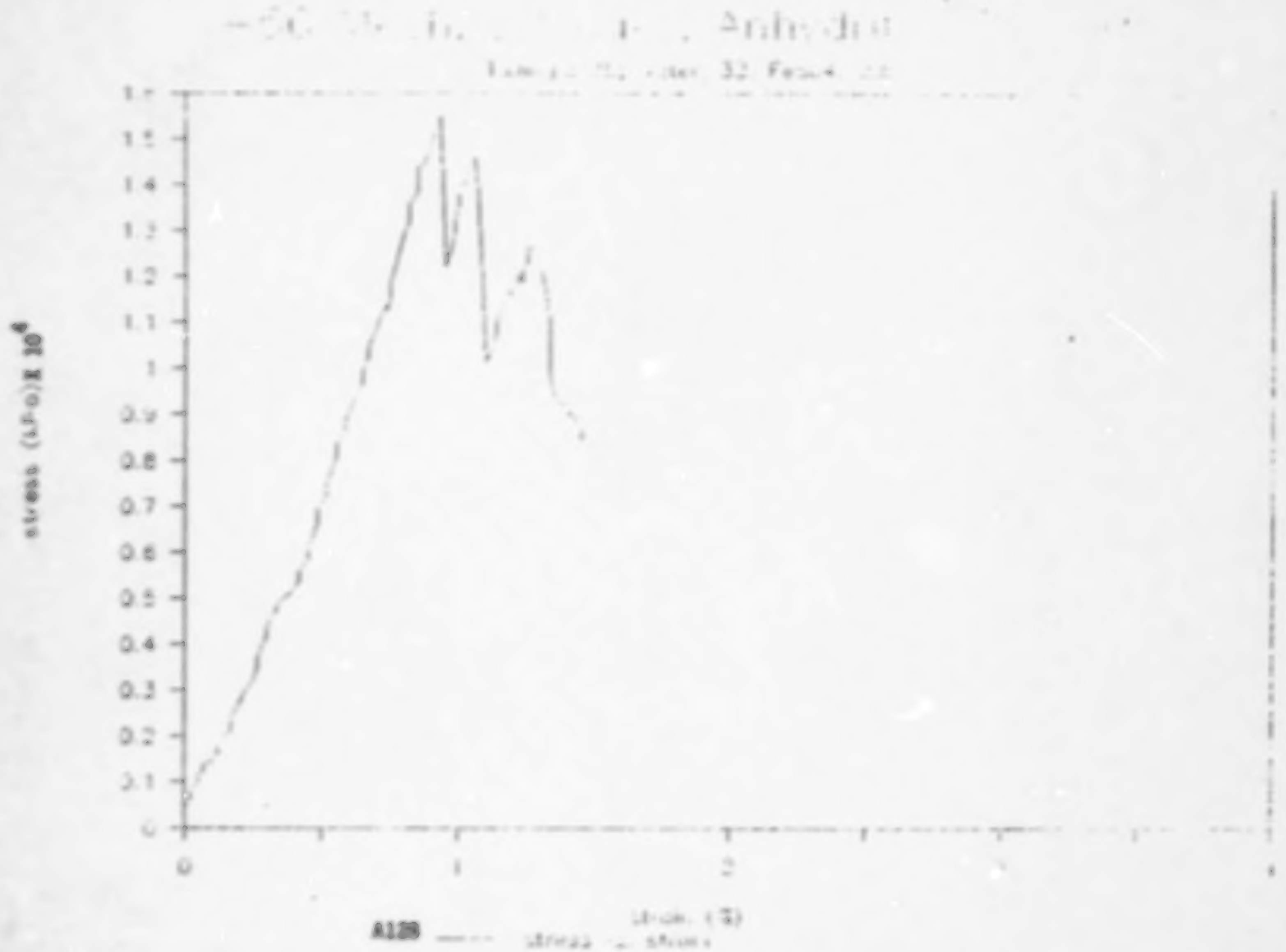
Tailings: 50; Water: 42; FeSo4: 3.8

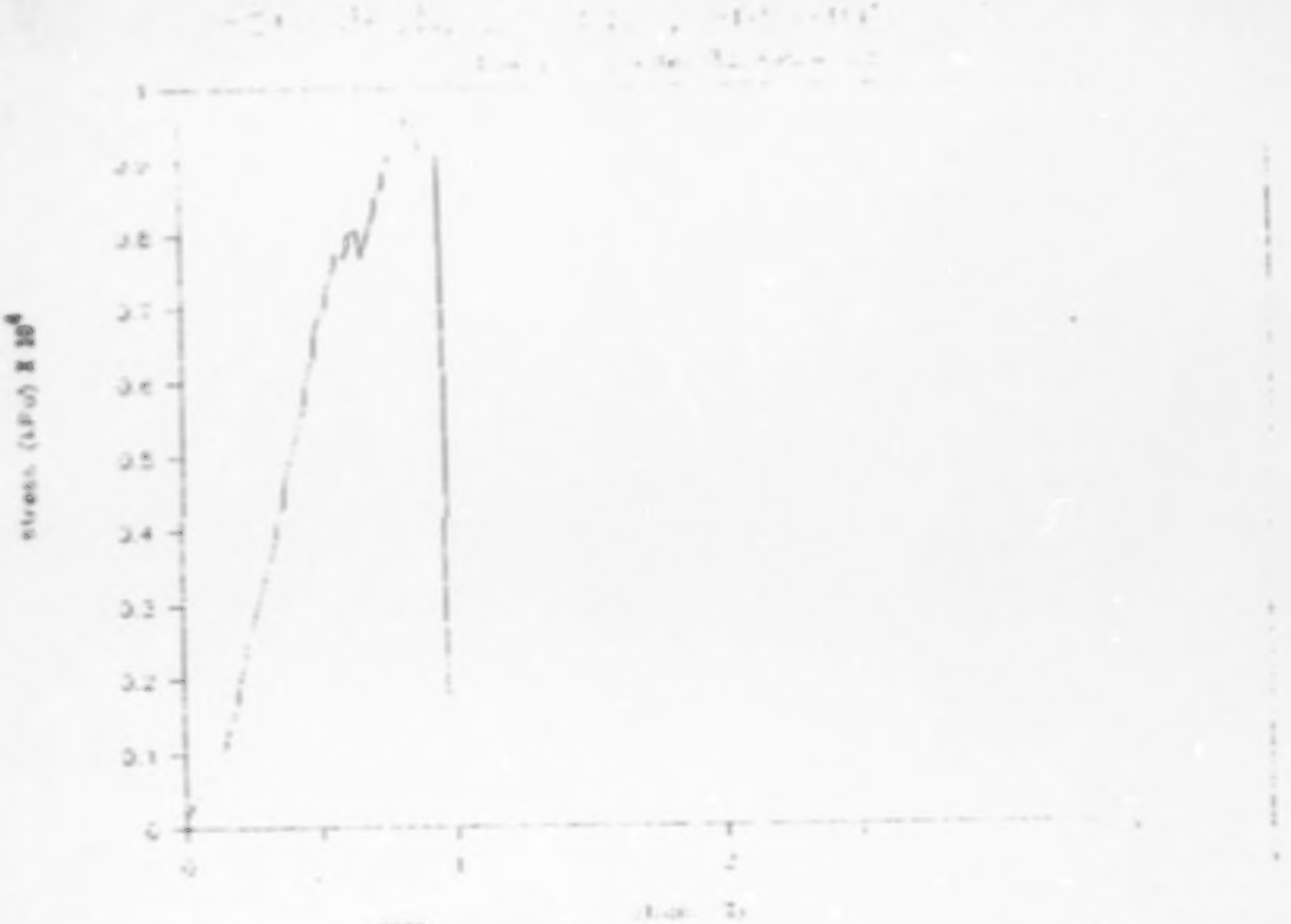



-50 Mesh; 14 days; Anhydrite: 25 parts
Tailings: 75; Water: 42; FeSo4: 3.6
stress (kPa) X 10^4
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A126 —— stress vs. strain

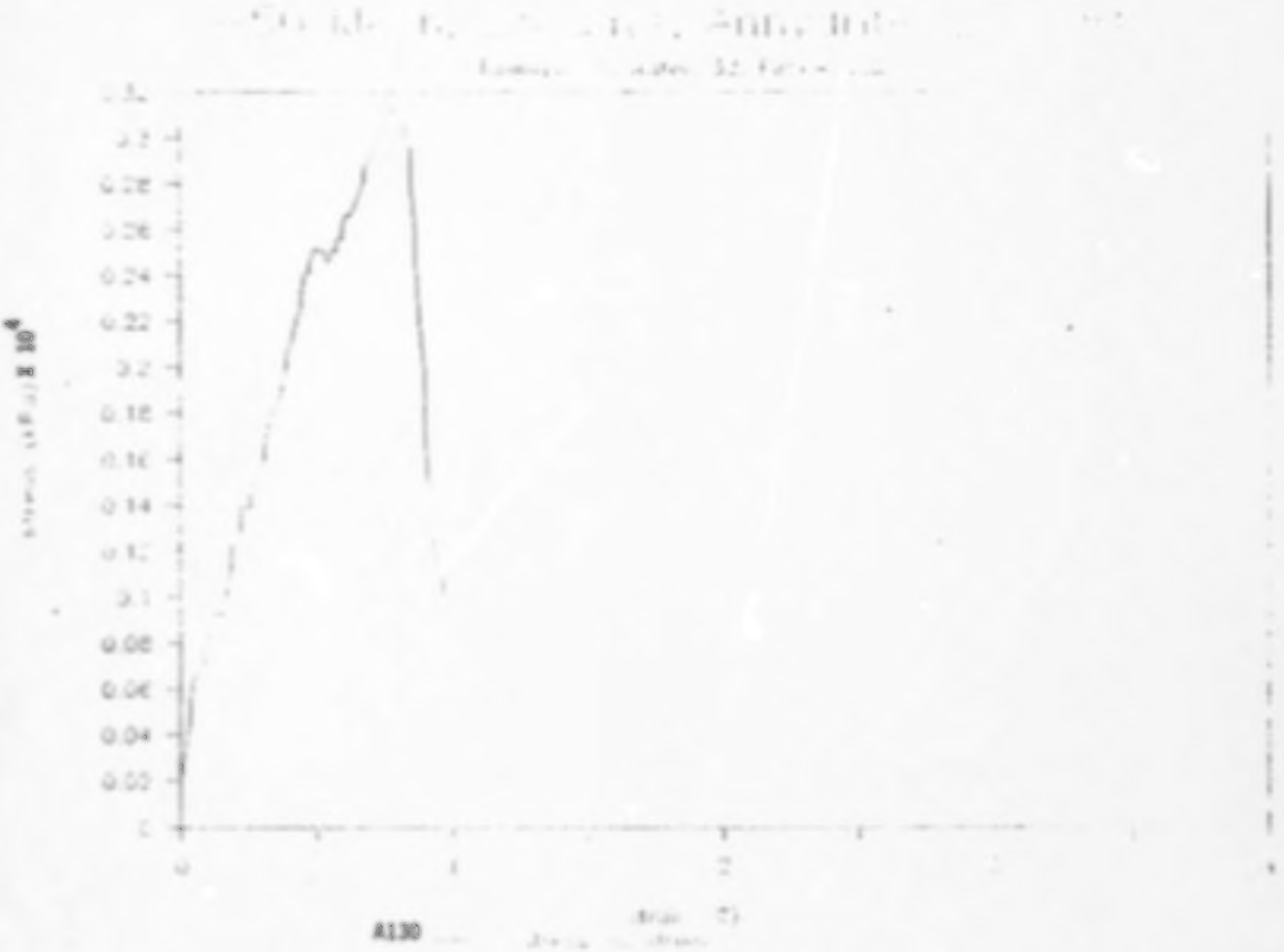


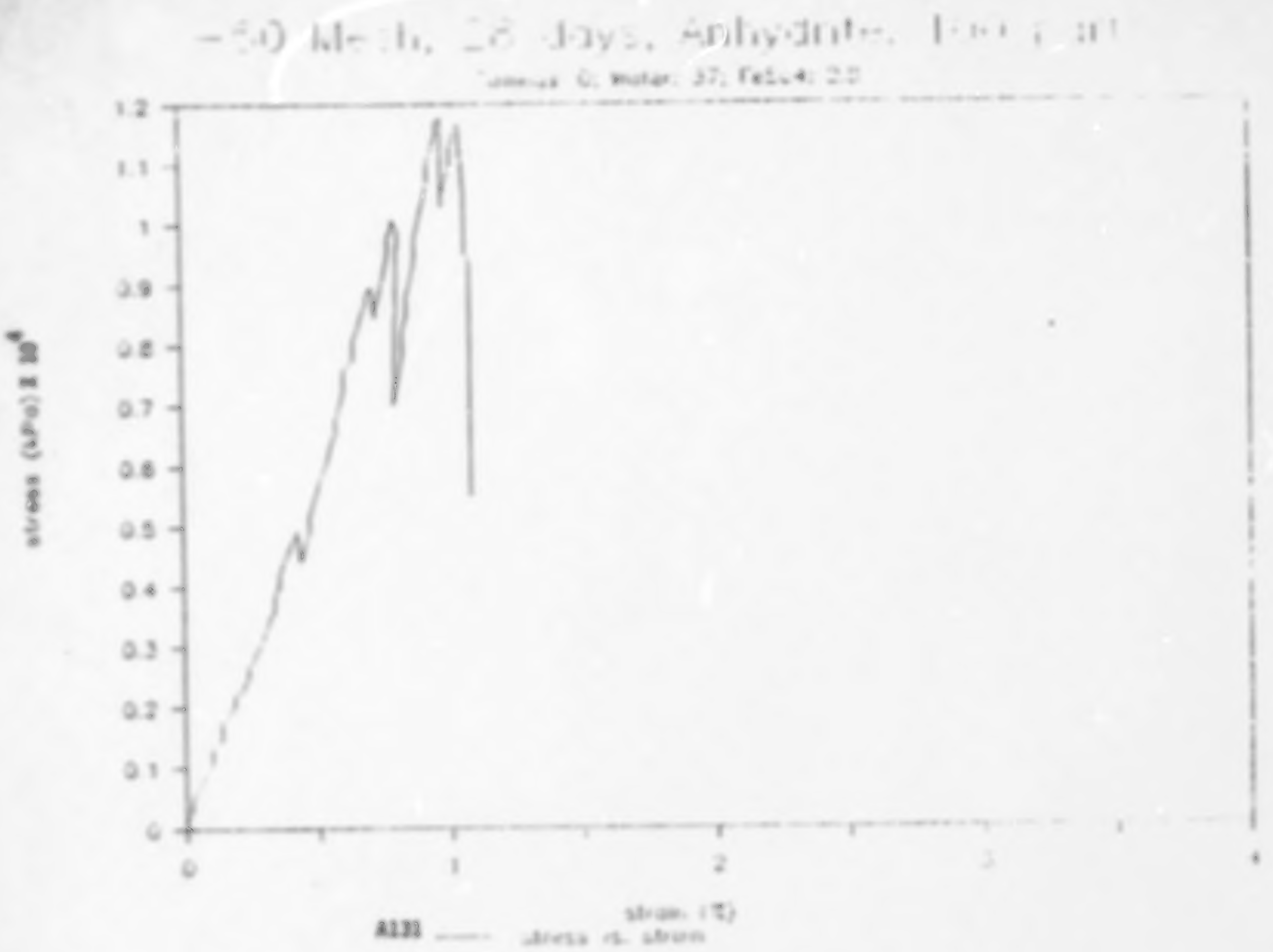
A127


stress (kPa)X 10^6
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A128
stress – strain



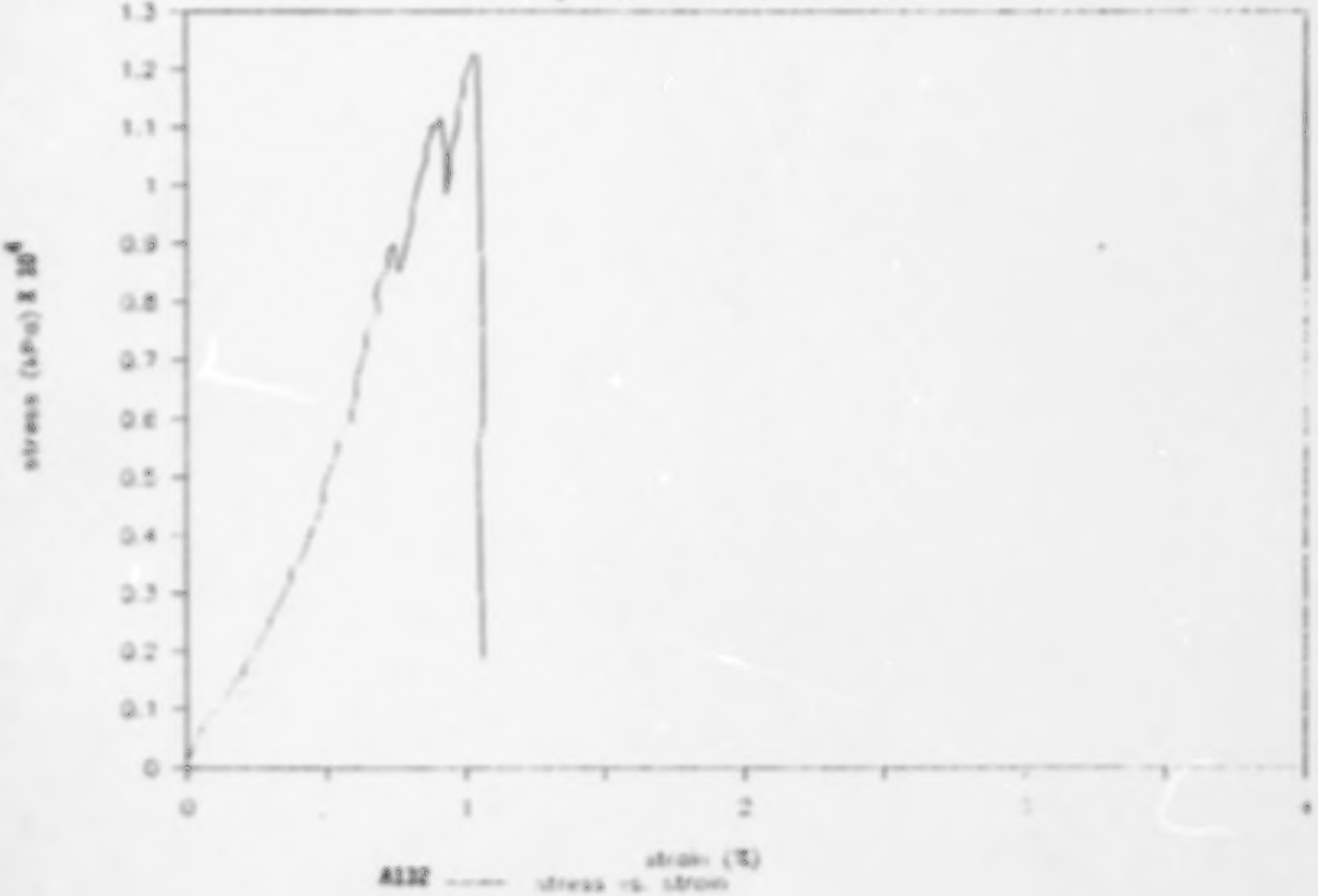
A129

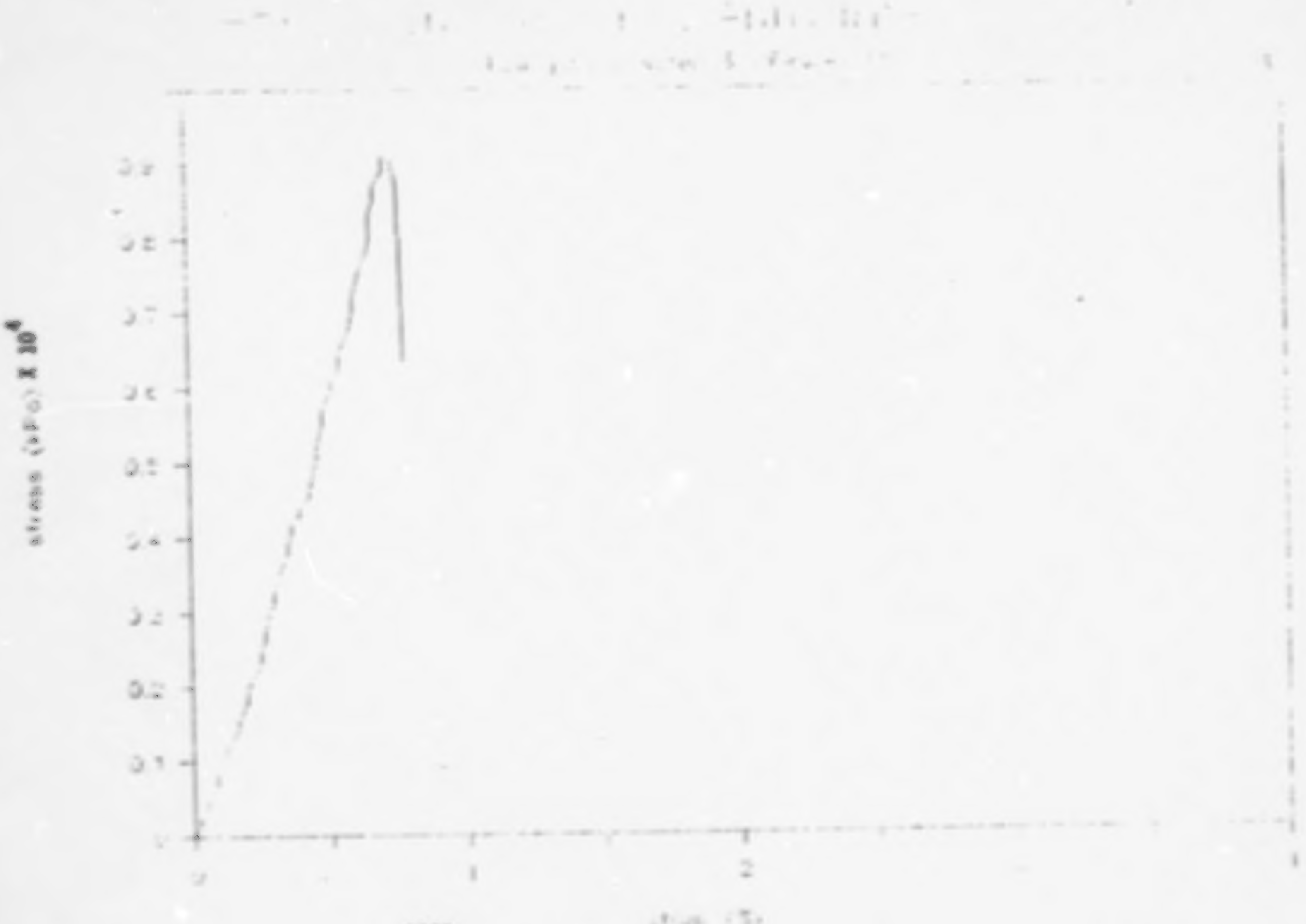

0.3
0.28
0.26
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
4
A130


-50 Mesh, 28 Days, Anhydrite, [illegible]
Cement: 0, Water: 37, FeSO4: 2.0
stress (kPa) X 10^4
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A133 —— stress vs. strain

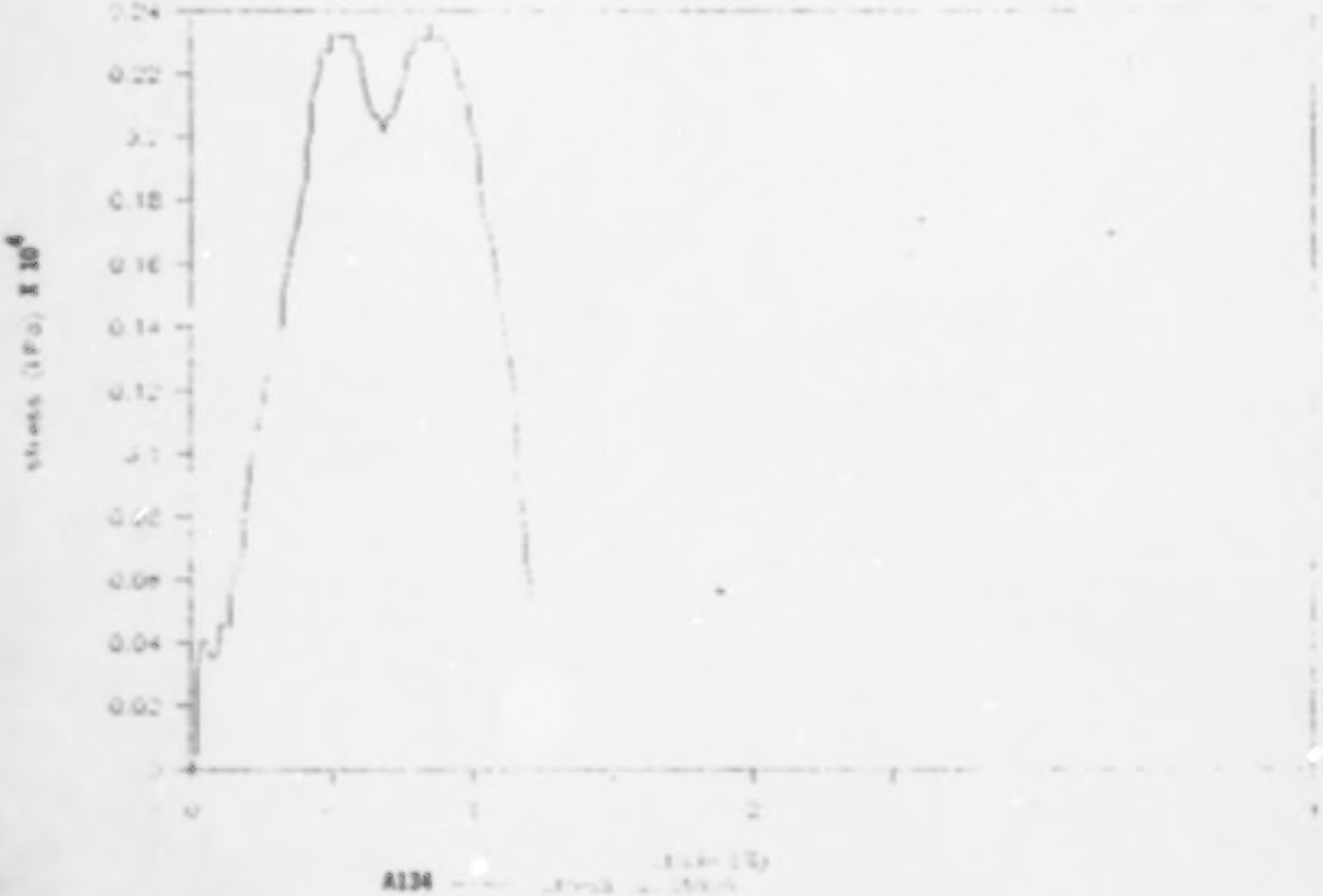
−50 Mesh; 28 days; Anhydrite: 75 parts

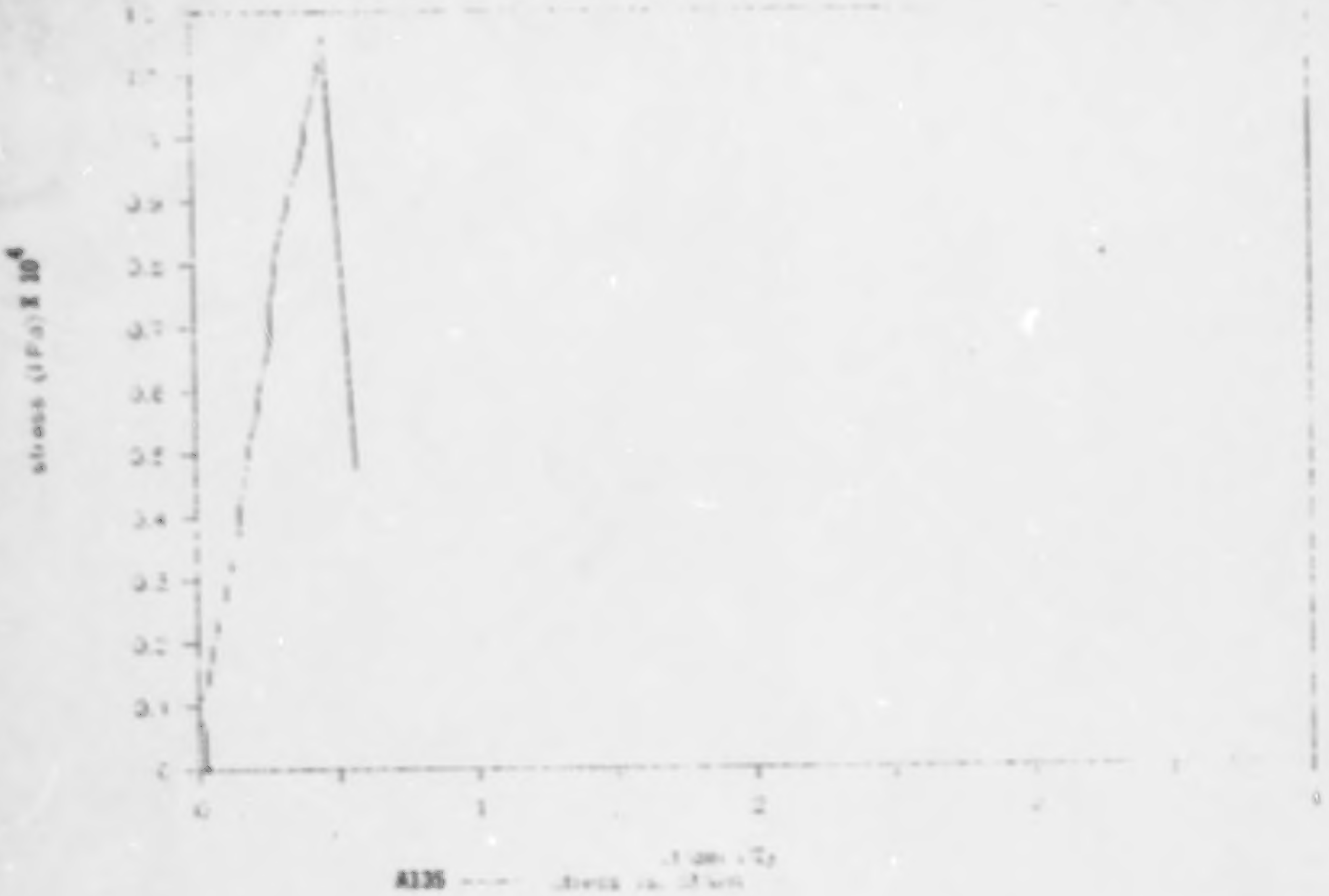
Tailings: 25; Water: 37; FeSO4: 2.8

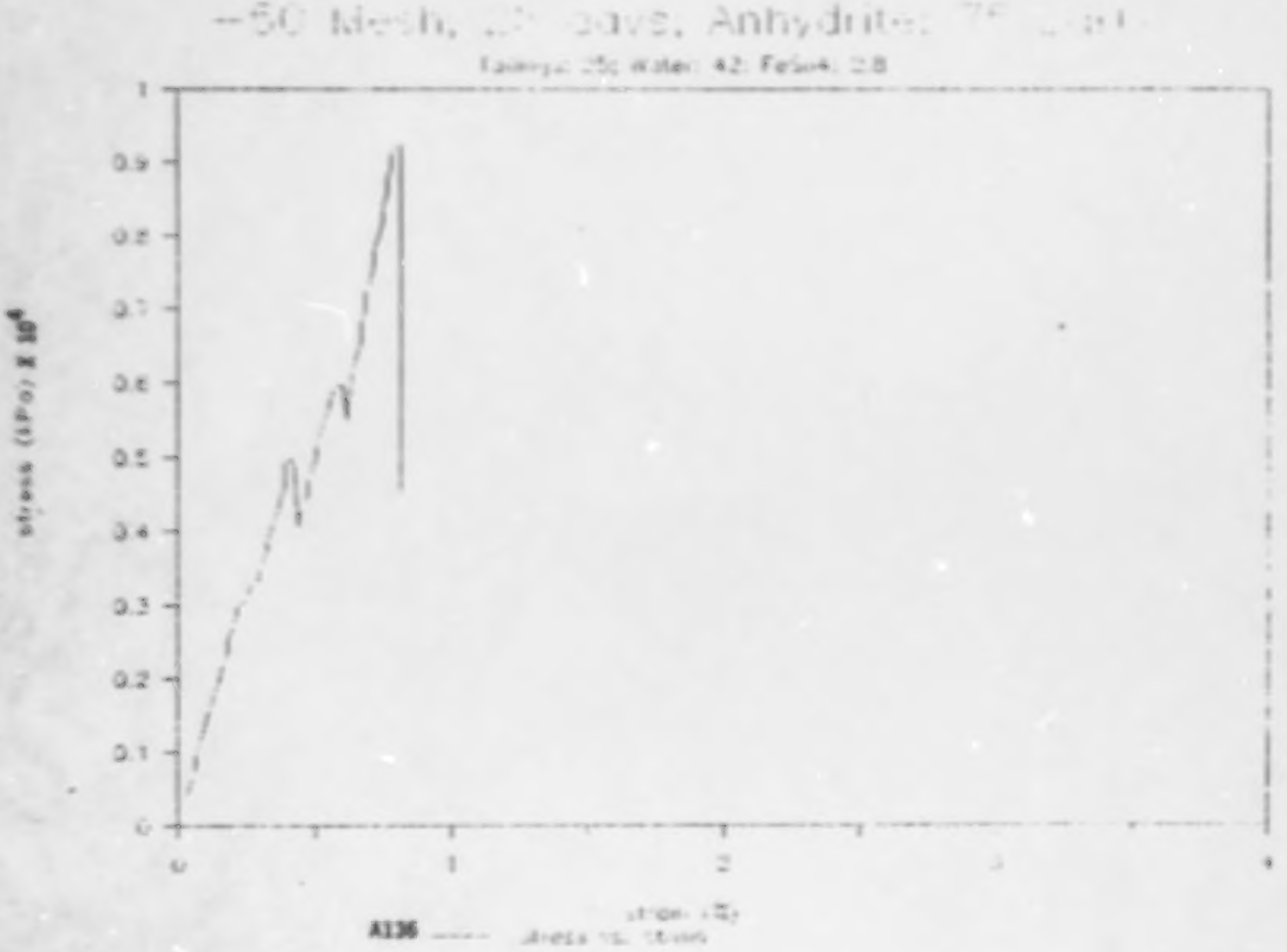


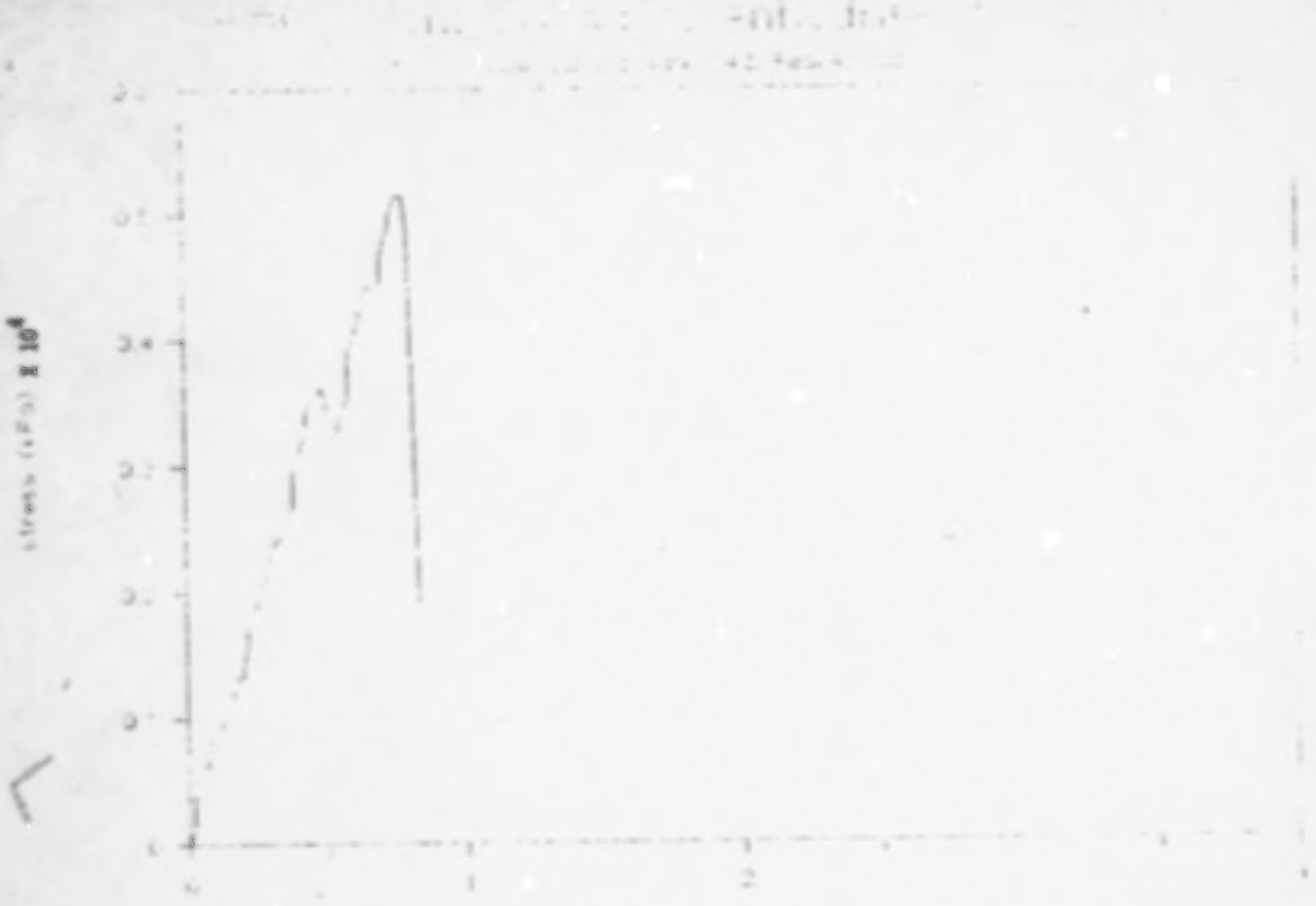


A133

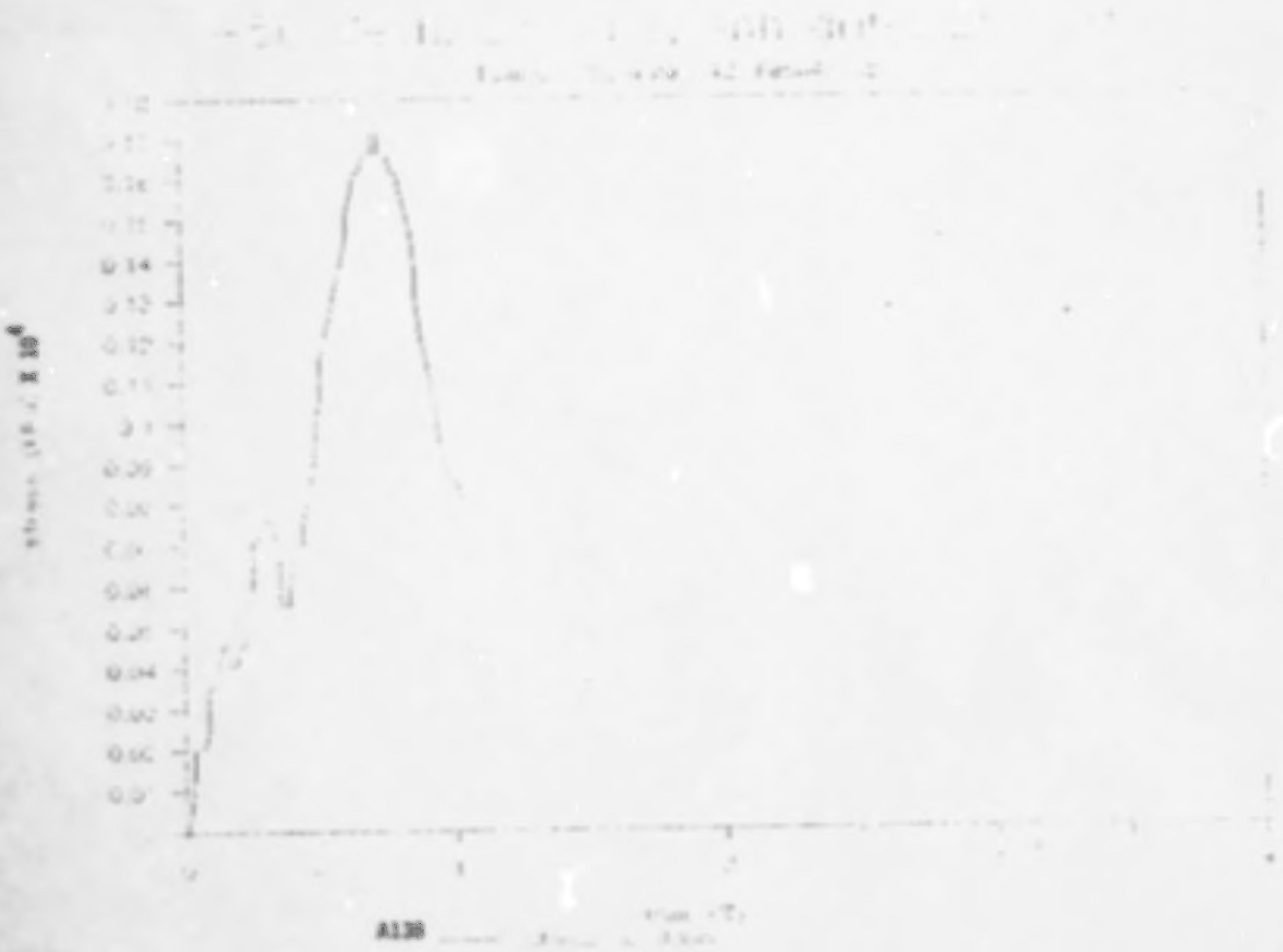

Water 37
Stress (kPa) x 10^6
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
A136

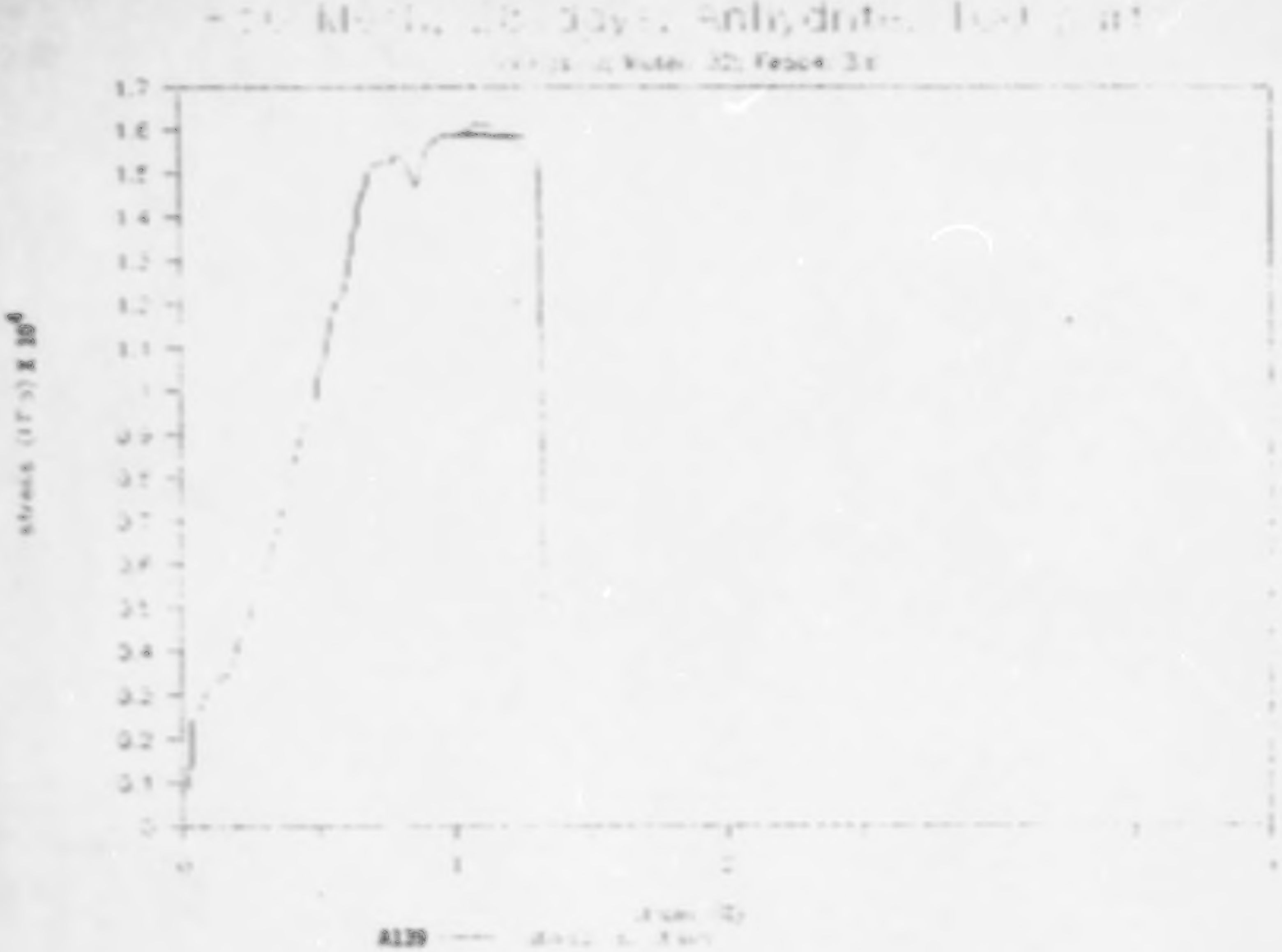

stress (MPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
0
1
2
3
4
A136 ---- Stress vs. Strain

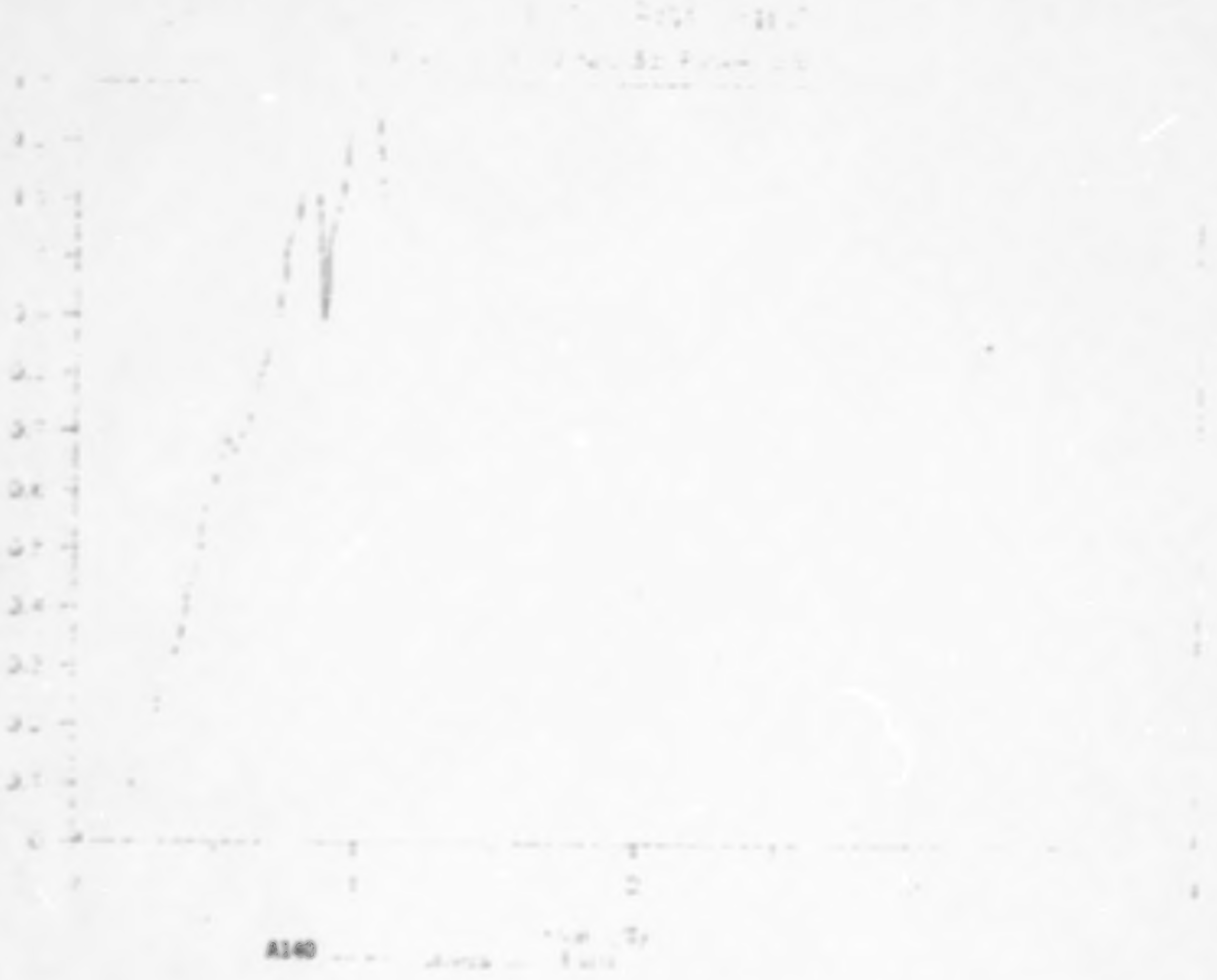

−50 Mesh, 2[illegible] Days, Anhydrite: 75 [illegible]
[illegible]: 25; Water: 42; FeSO4: 2.8
Stress (kPa) X 10⁴
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
Strain (%)
A136
Stress vs. Strain



A137 ---- Stress vs Strain


0.14
0.04
0.02
X 10^6
1
2
A138

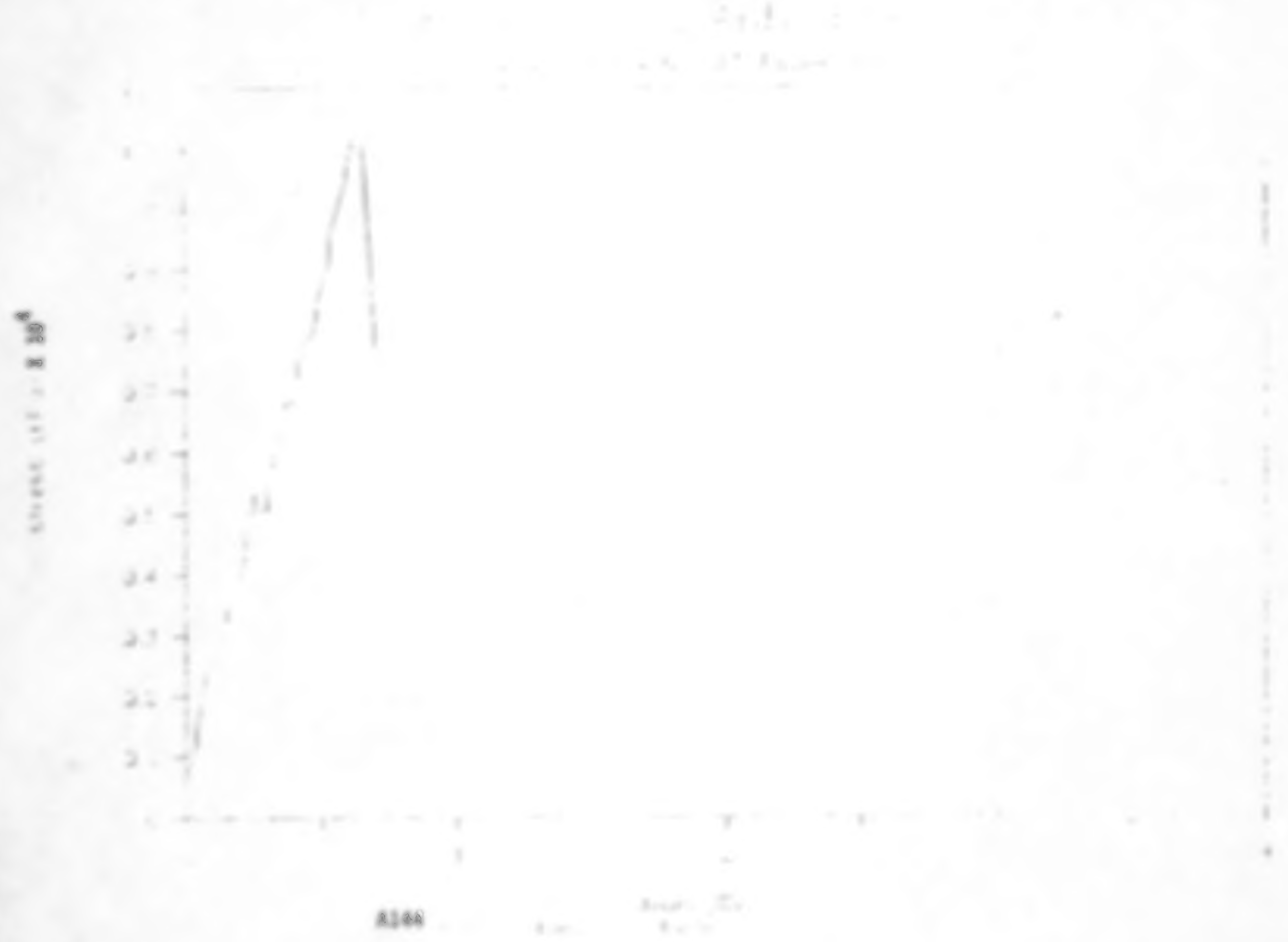

Water 32, Fesco 3x
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
Stress
A139

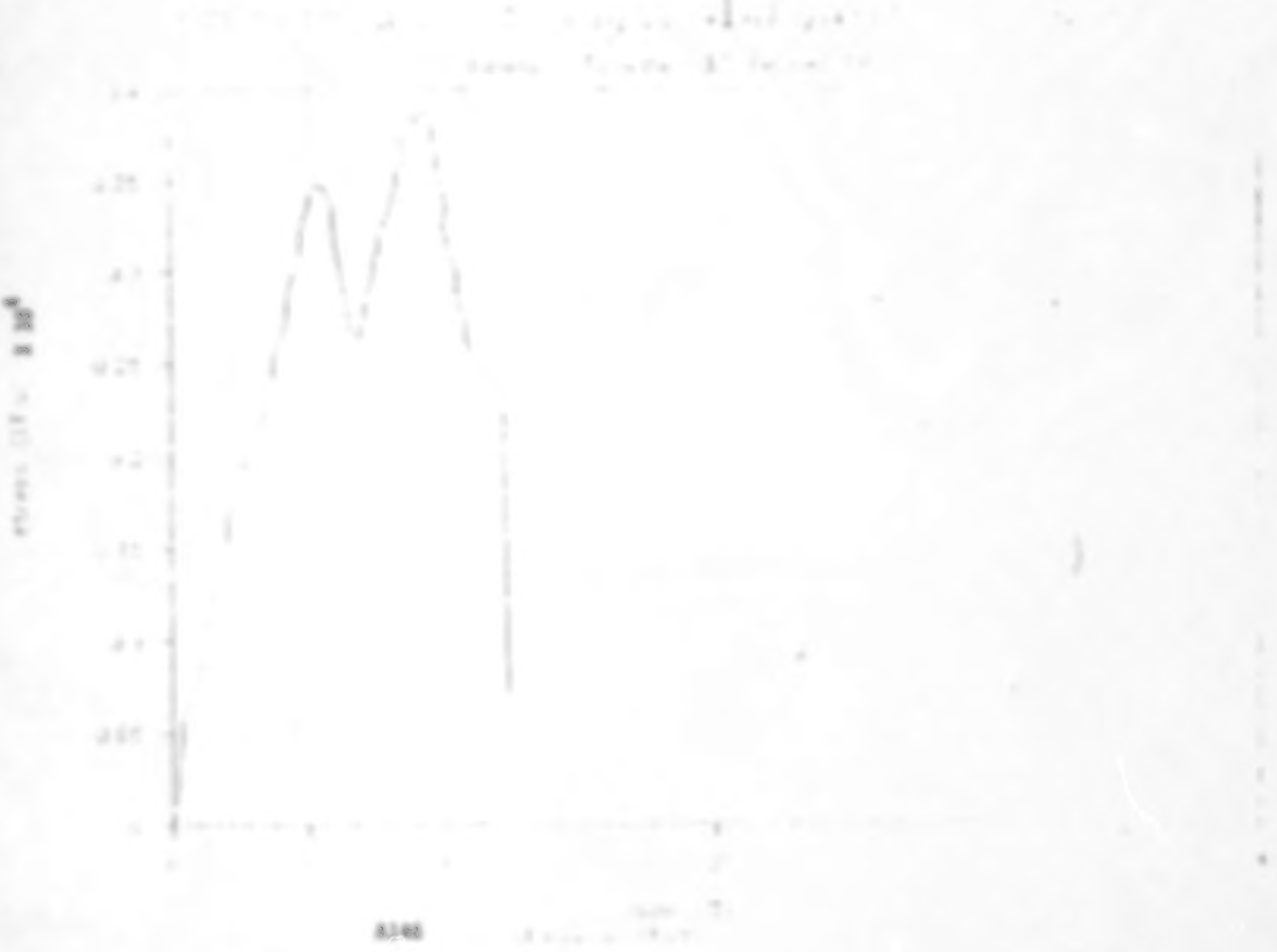

A140

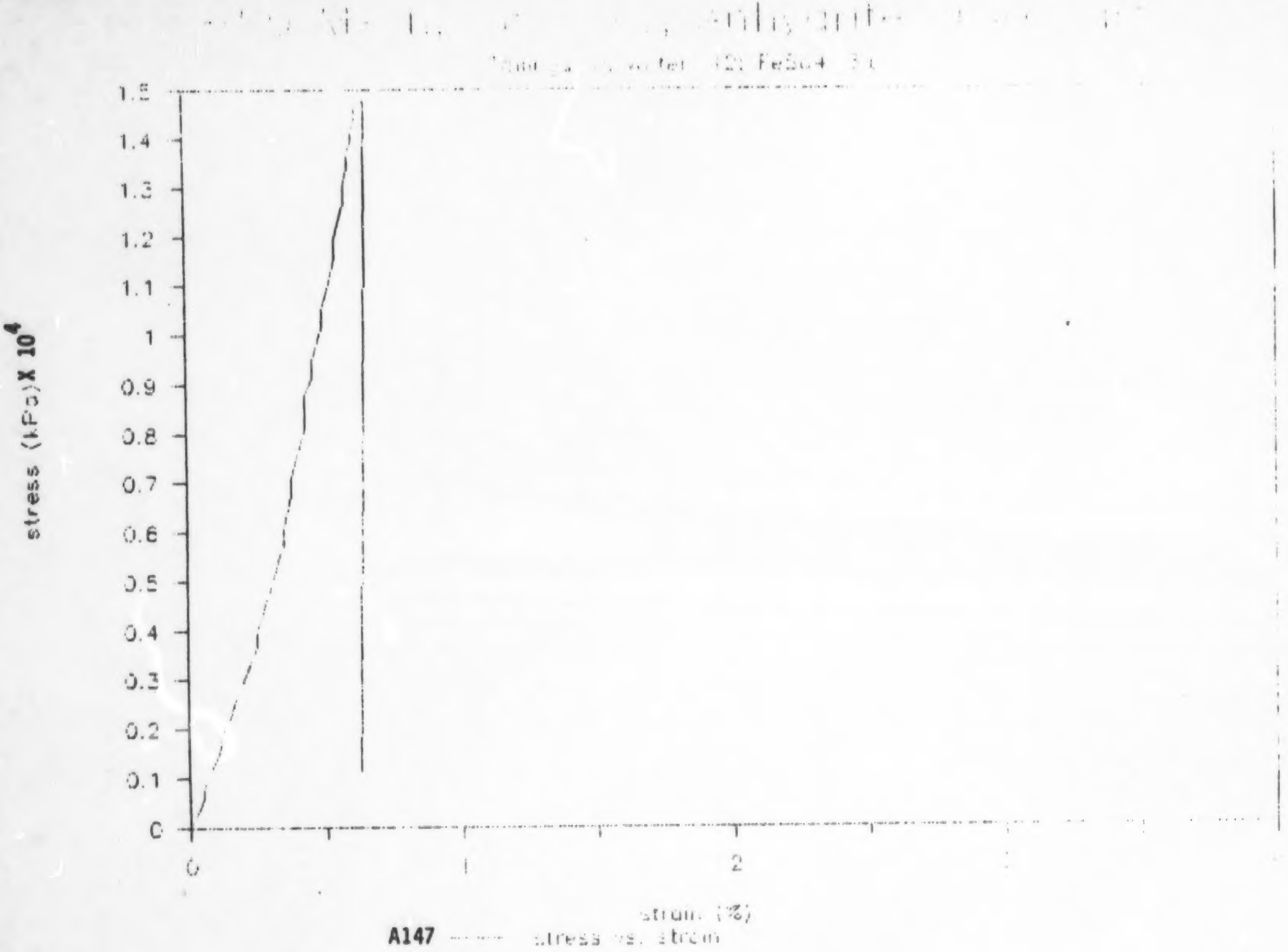
A361

A142

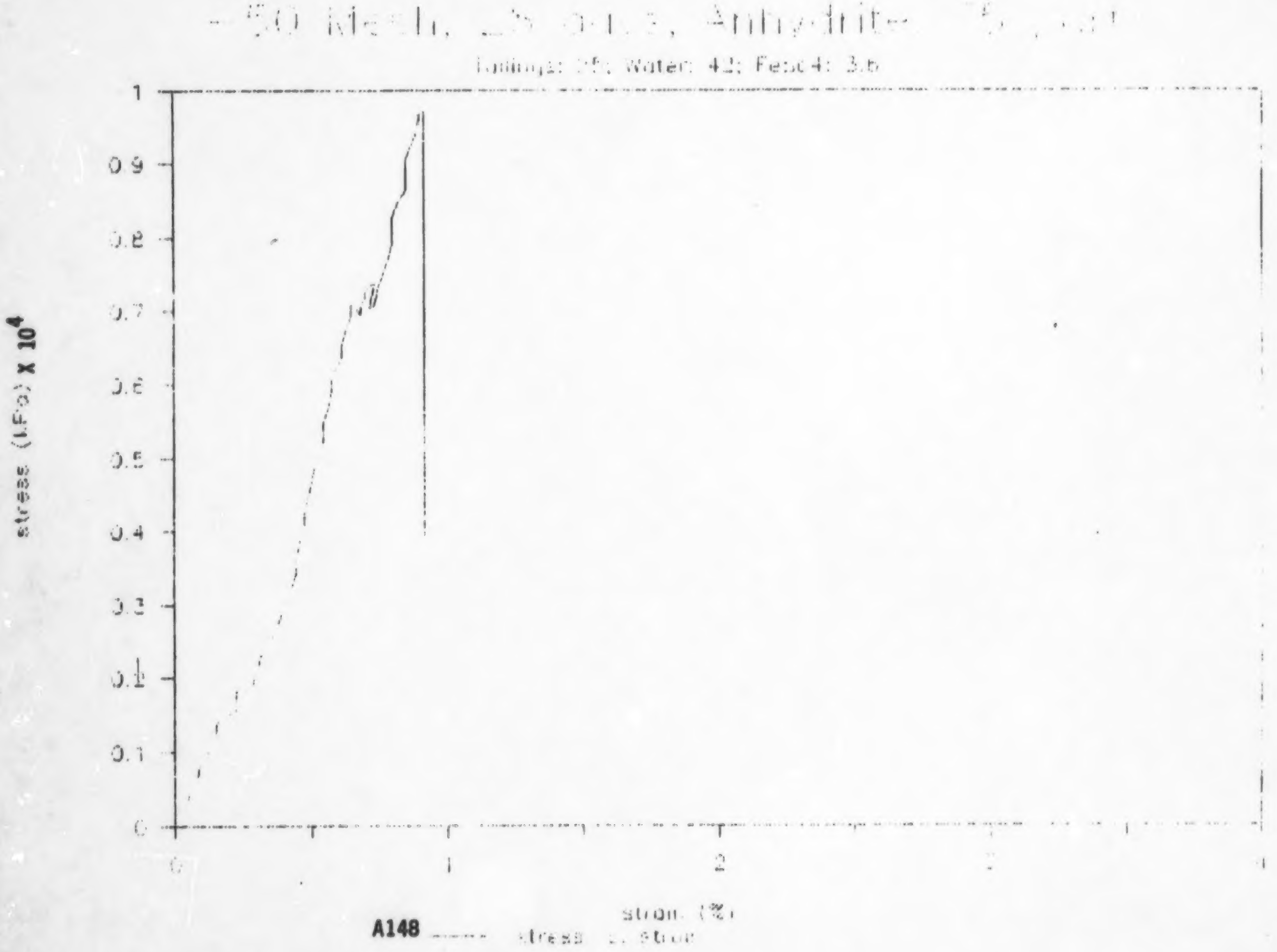
A563
x 10^4

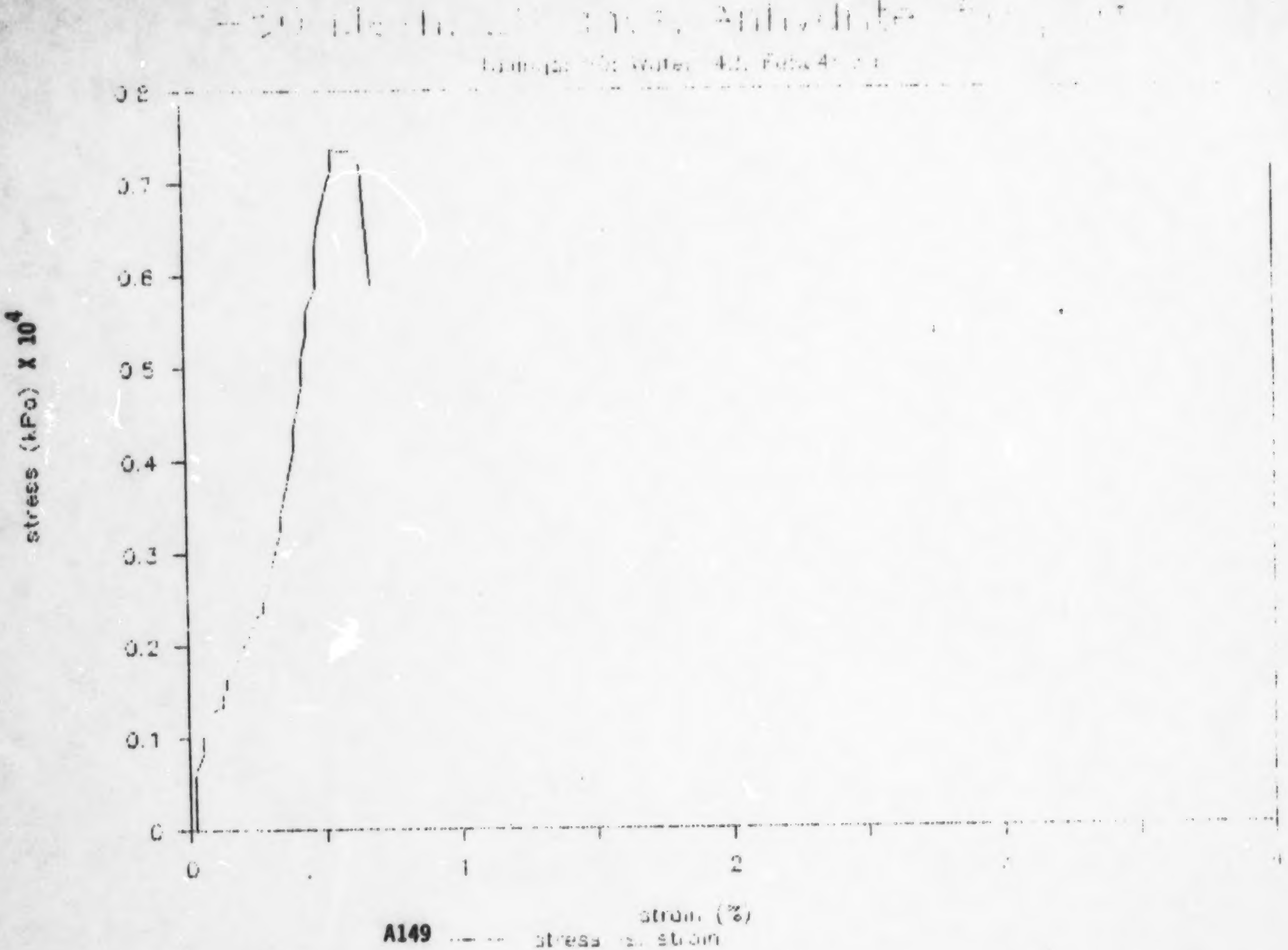

X 10^4
A144


X 10^6
A348



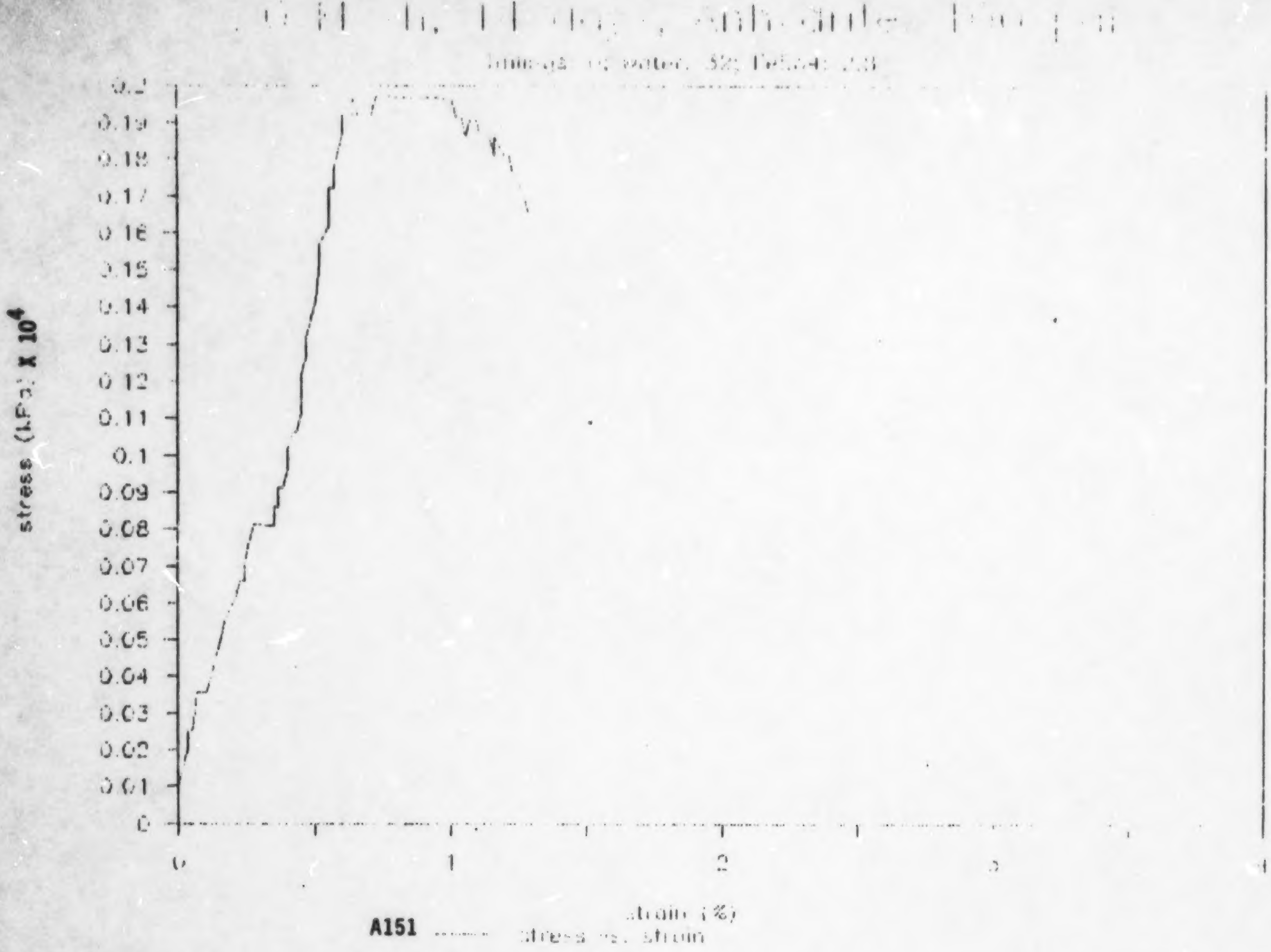
stress (kPa) X 10^4
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
0
1
2
strain (%)
A147 stress vs. strain


-50 Mesh, [illegible], Anhydrite [illegible]
Tailings: 25; Water: 42; FeSO4: 3.6
stress (LFo) X 10^4
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A148
stress vs. strain


0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
stress (kPa) X 10^4
0
1
2
strain (%)
A149 —— stress vs. strain

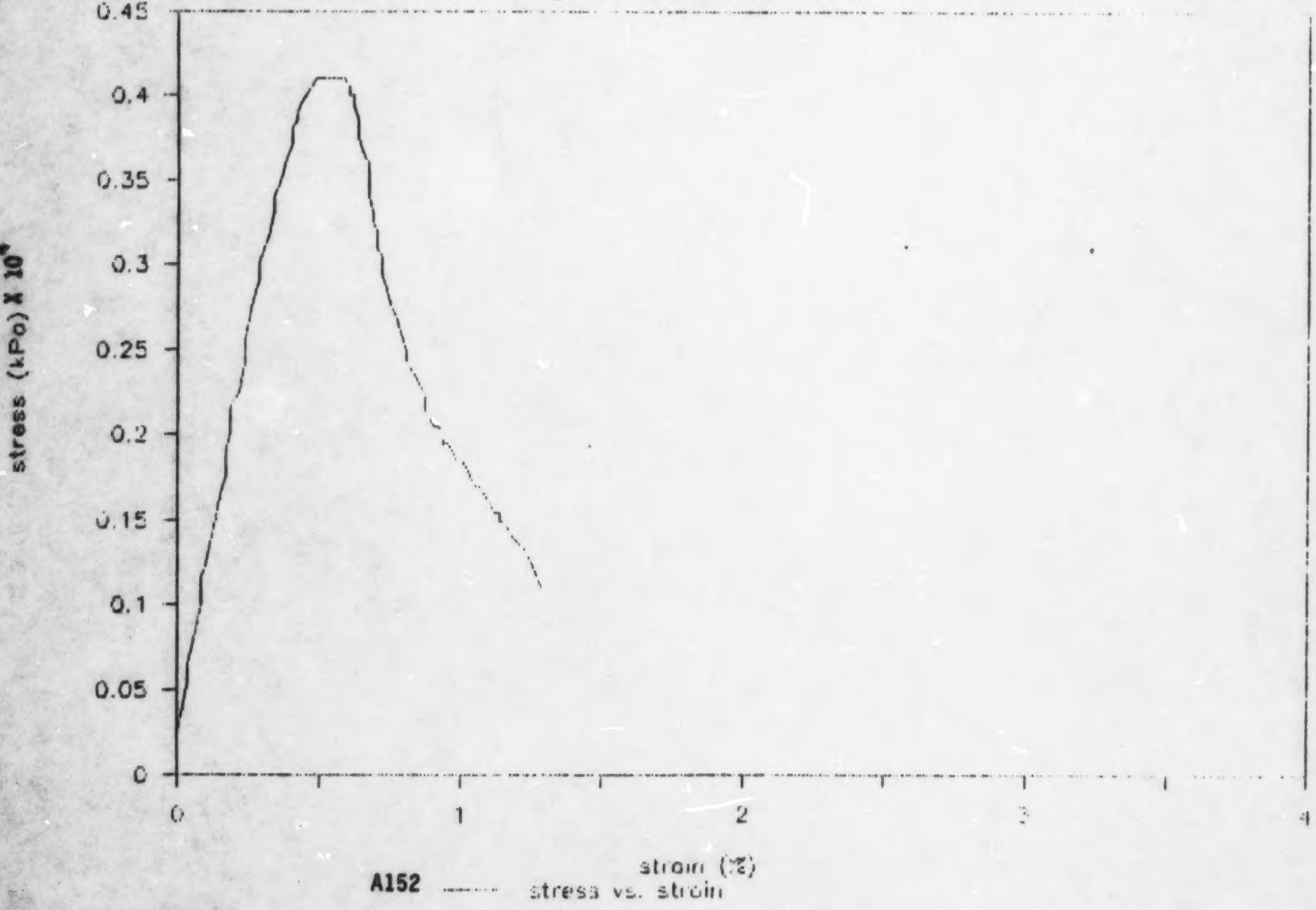
stress (LFS) X 10^4
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2

A150


stress (kPa) X 10^4
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A151
stress vs. strain

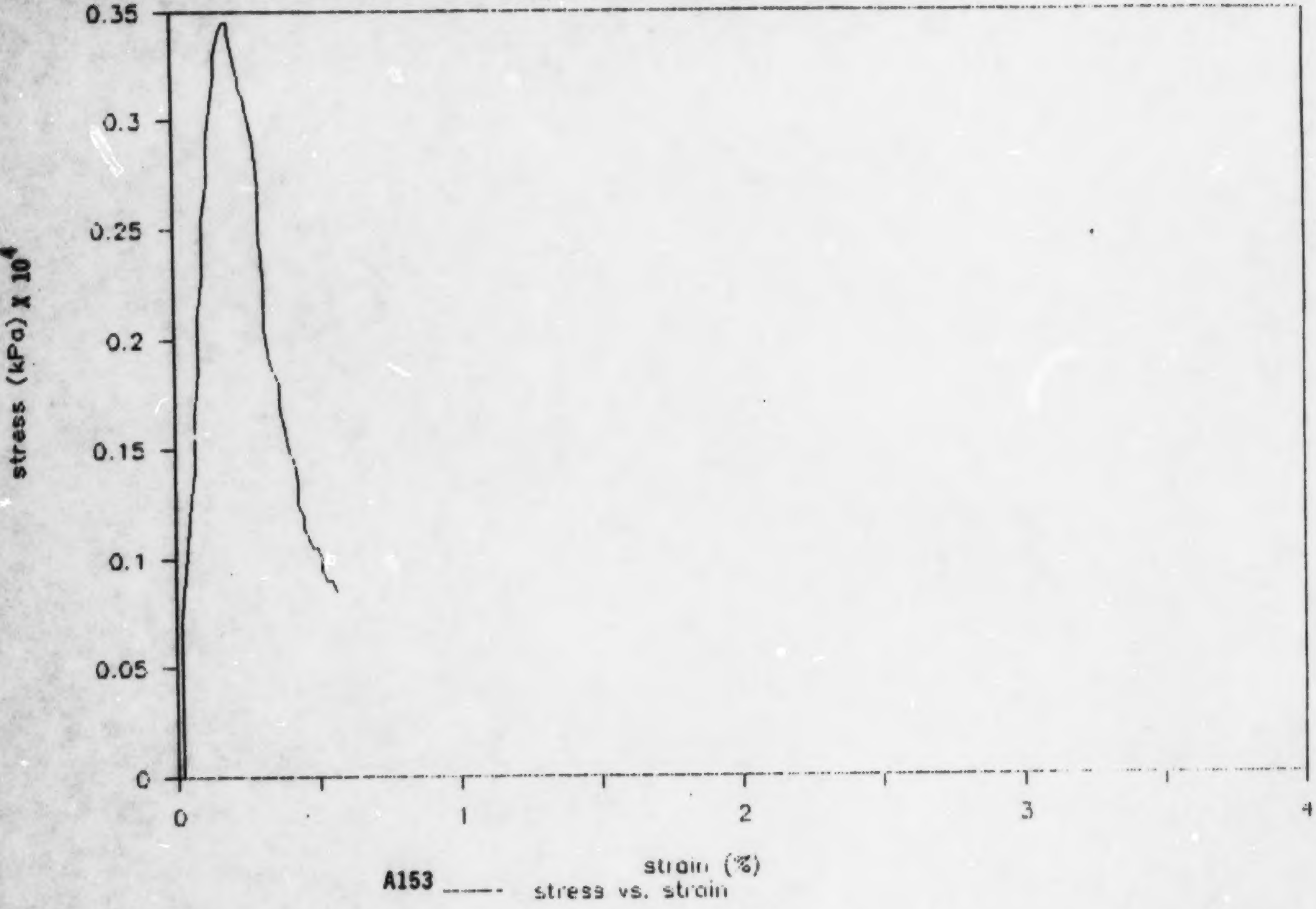
-20 Mesh, 14 days; Anhydrite: 75 parts

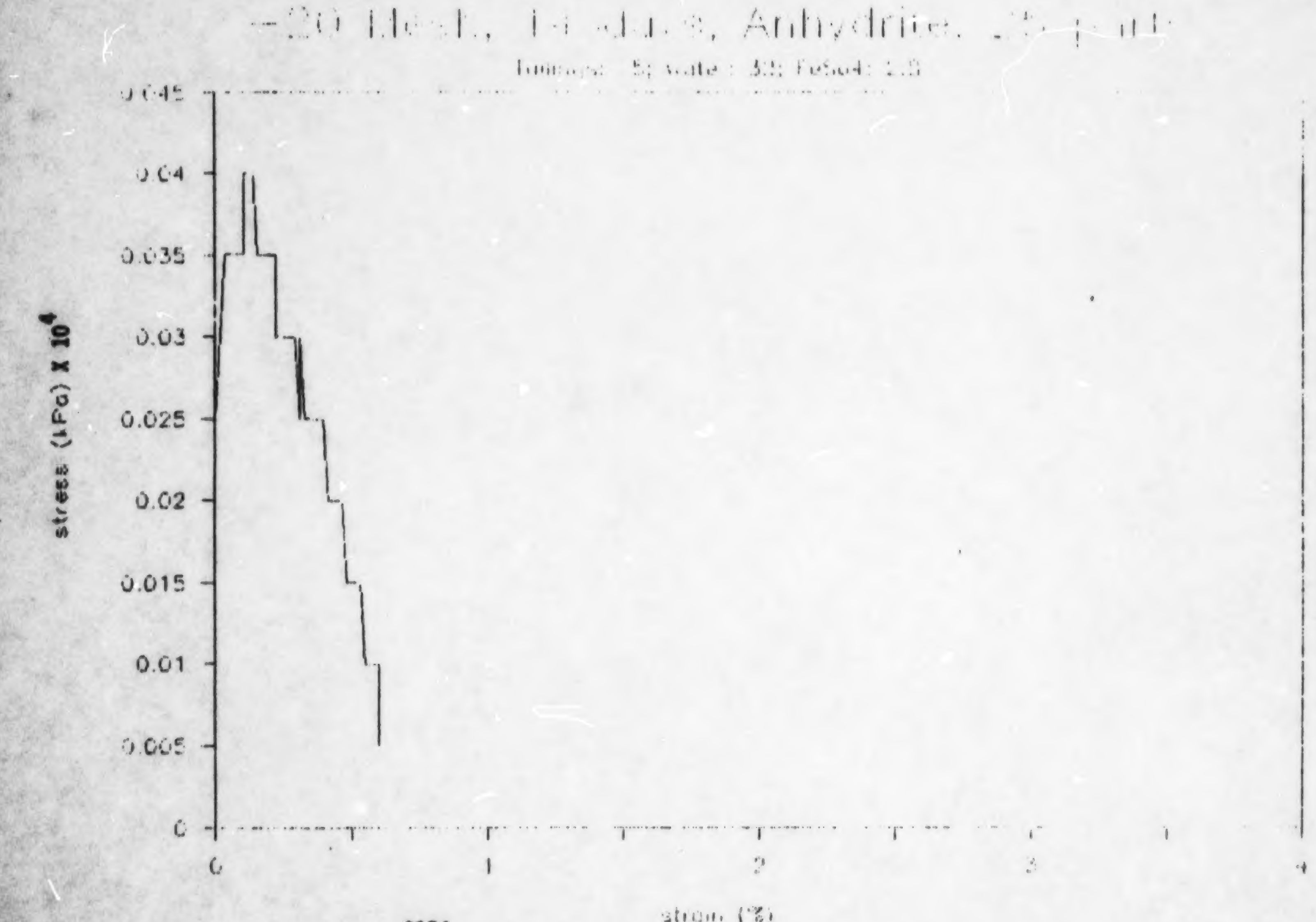
Tailings: 25; Water: 32; FeSo4: 2.8



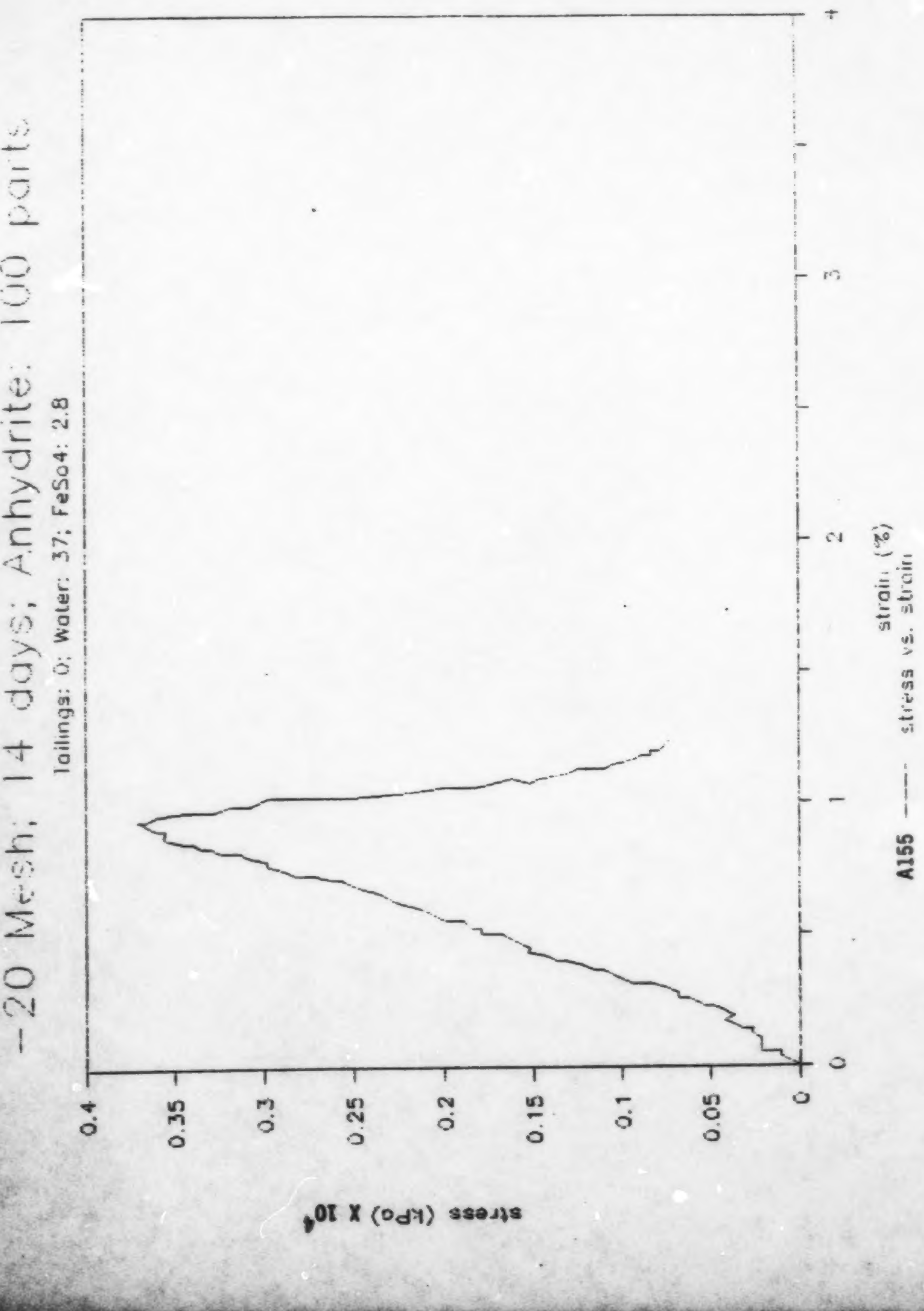
−20 Mesh; 14 days; Anhydrite: 50 parts

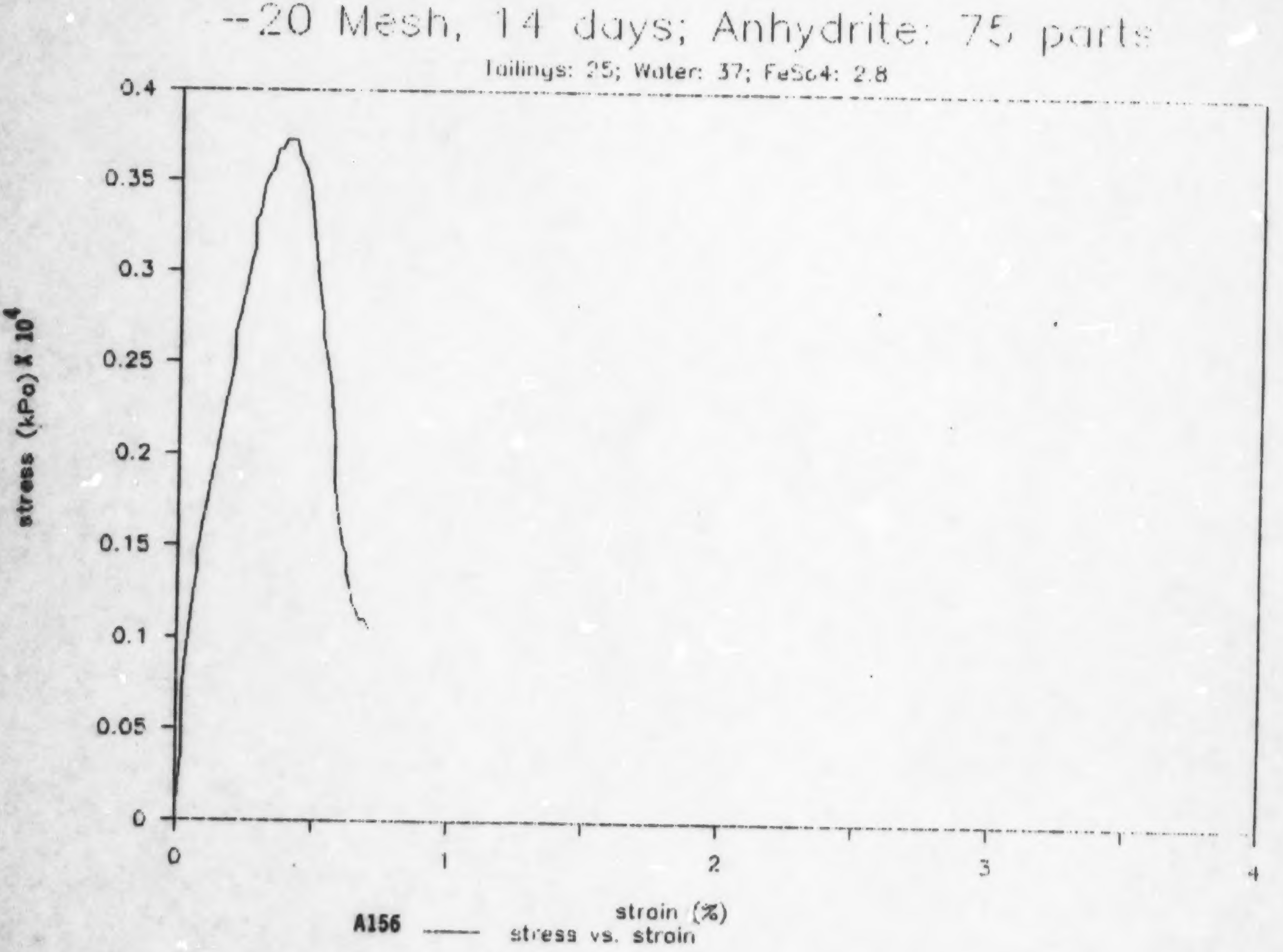
Tailings: 50; Water: 32; FeSo4: 2.8

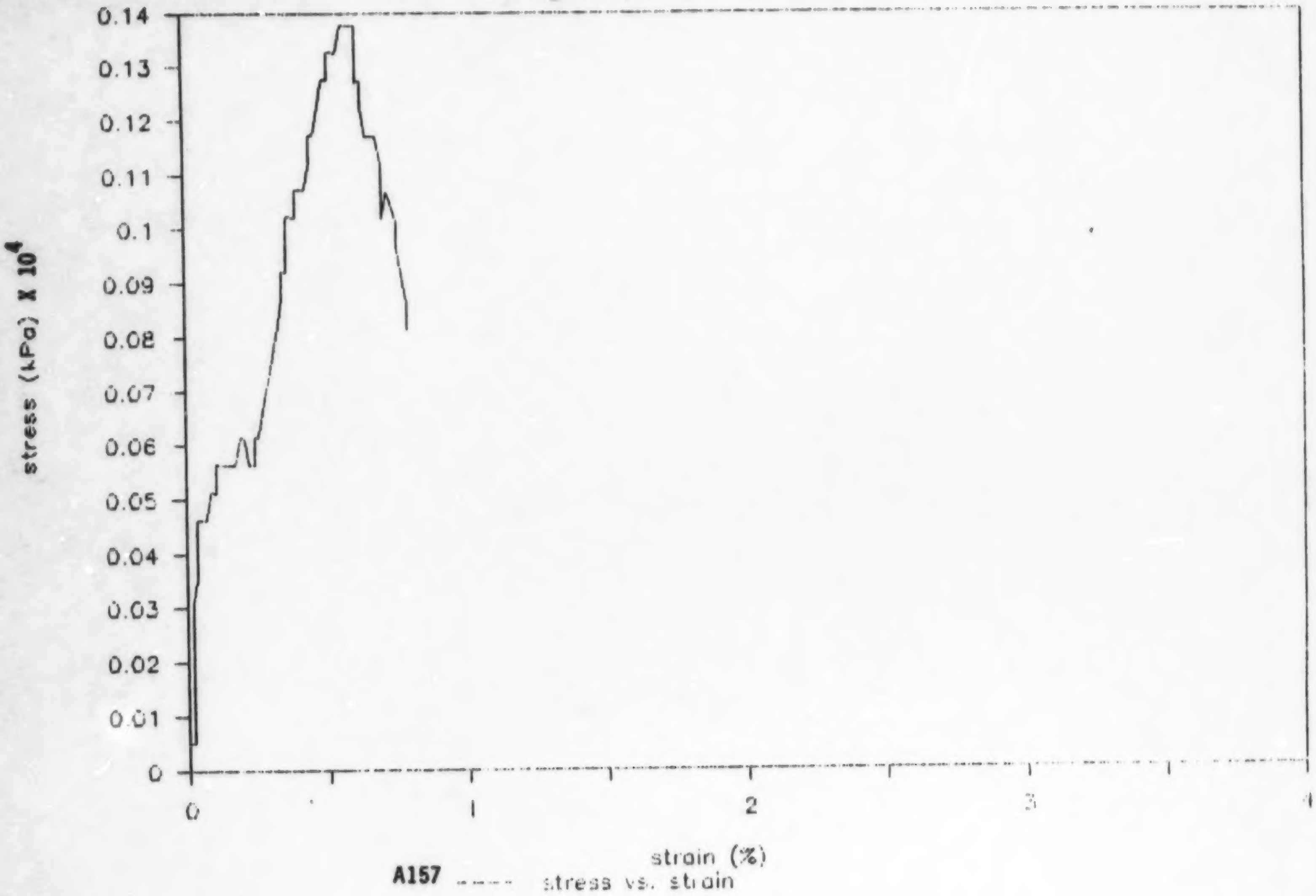



-20 Mesh, 14 days, Anhydrite
stress (kPa) X 10^4
0.045
0.04
0.035
0.03
0.025
0.02
0.015
0.01
0.005
0
0
1
2
3
4
strain (%)
stress vs. strain

A154

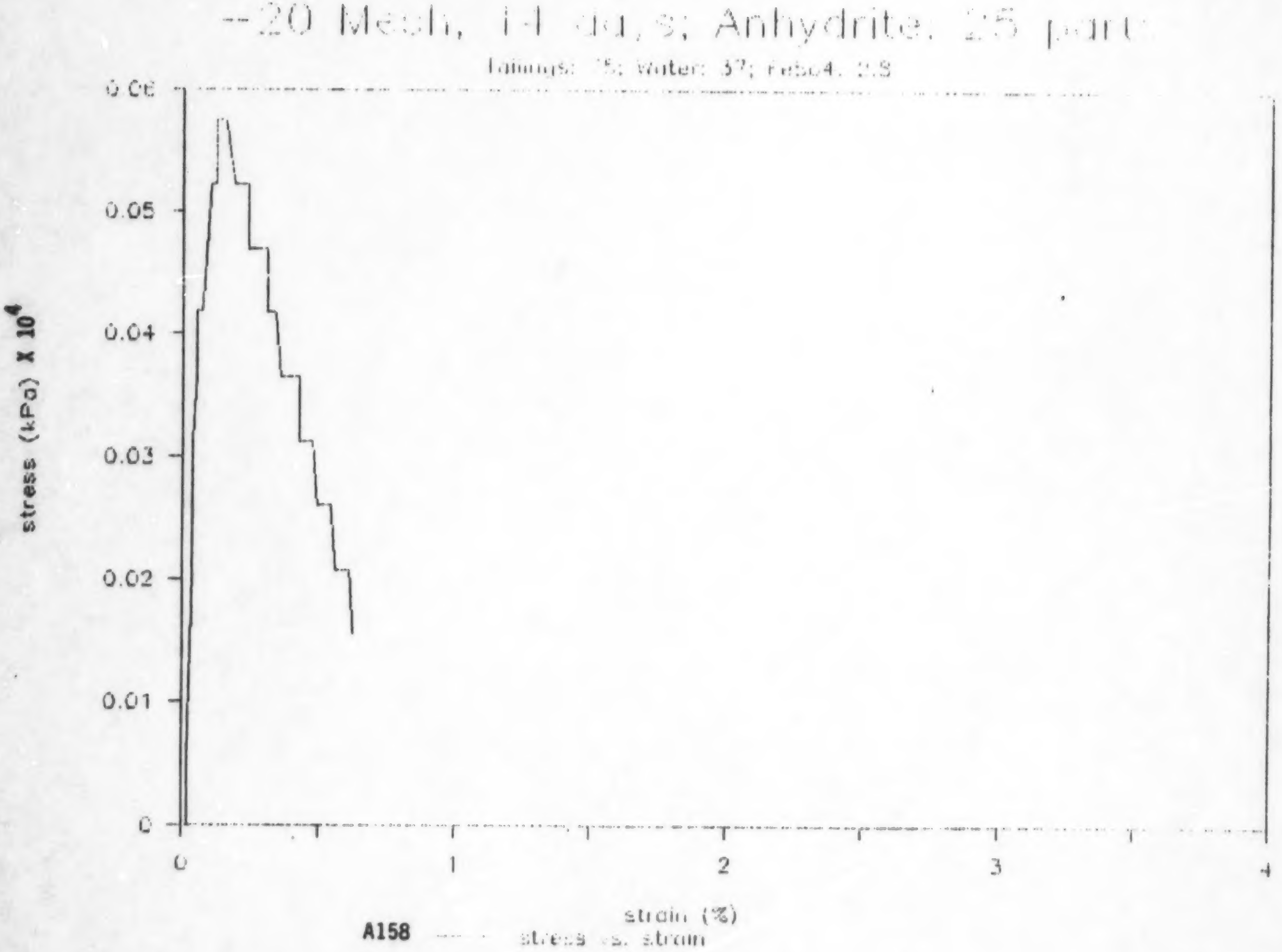

-20 Mesh; 14 days; Anhydrite: 100 parts
Tailings: 0; Water: 37; FeSo4: 2.8
stress (kPa) X 10^4
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
3
4
strain (%)
A155 —— stress vs. strain

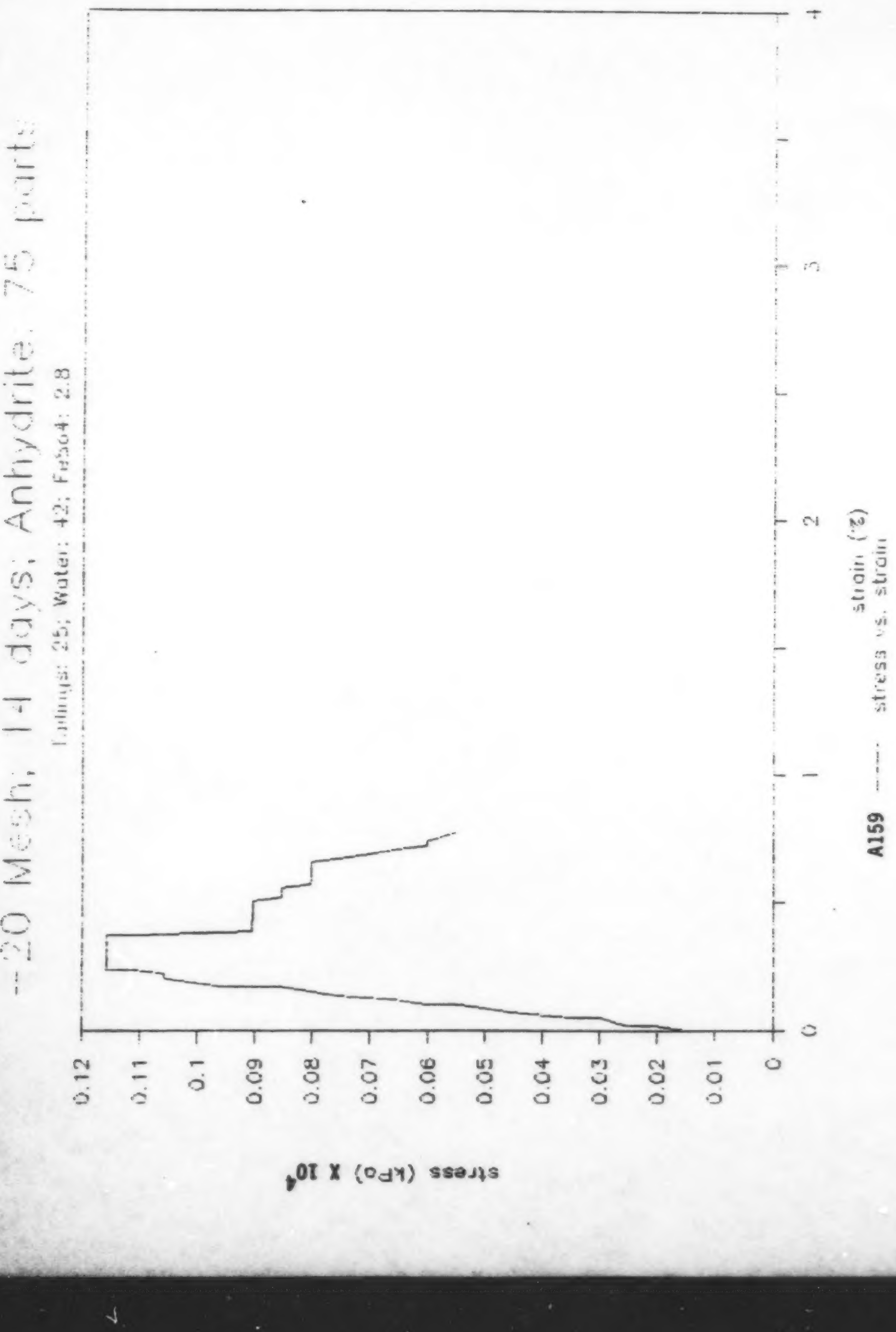

--20 Mesh, 14 days; Anhydrite: 75 parts
Tailings: 25; Water: 37; FeSo4: 2.8
stress (kPa) X 10^4
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
3
4
strain (%)
A156 —— stress vs. strain



−20 Mesh; 14 days; Anhydrite: 50 parts

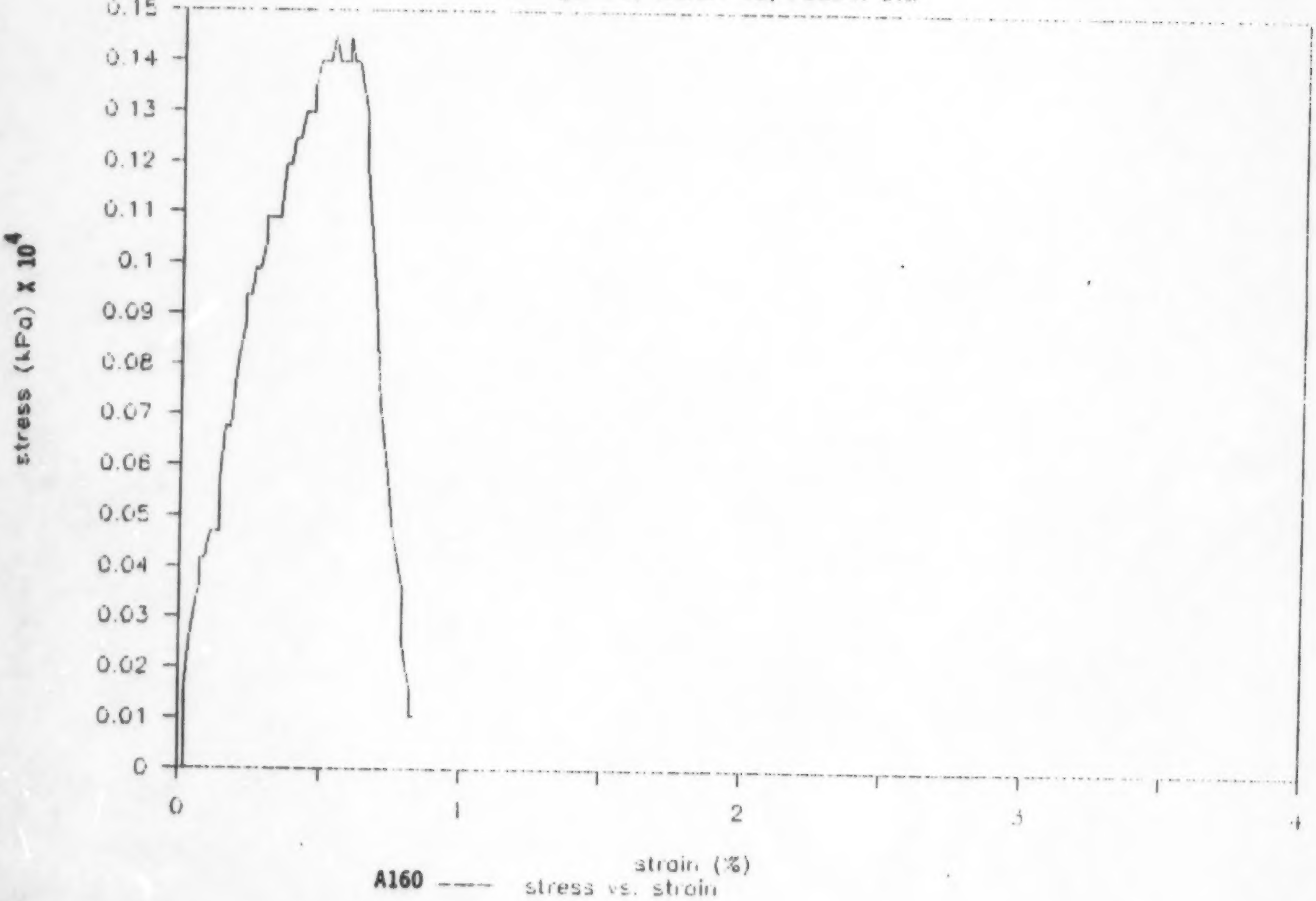
Tailings: 50; Water: 37; FeSo4: 2.8

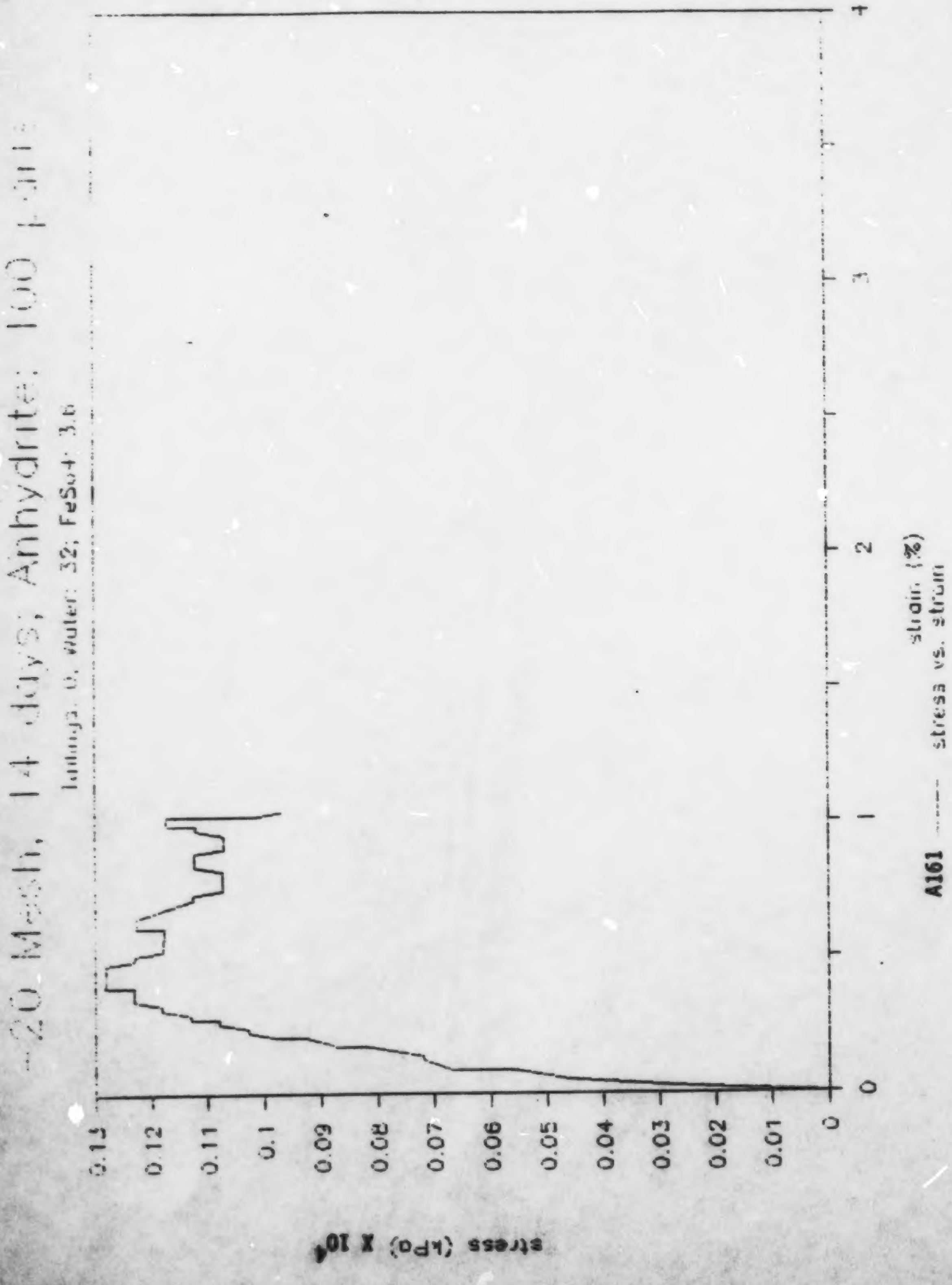

−20 Mesh, 14 days; Anhydrite: 25 parts
Tailings: 75; Water: 37; FeSO4: 2.8
stress (kPa) X 10⁴
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A158
stress vs. strain

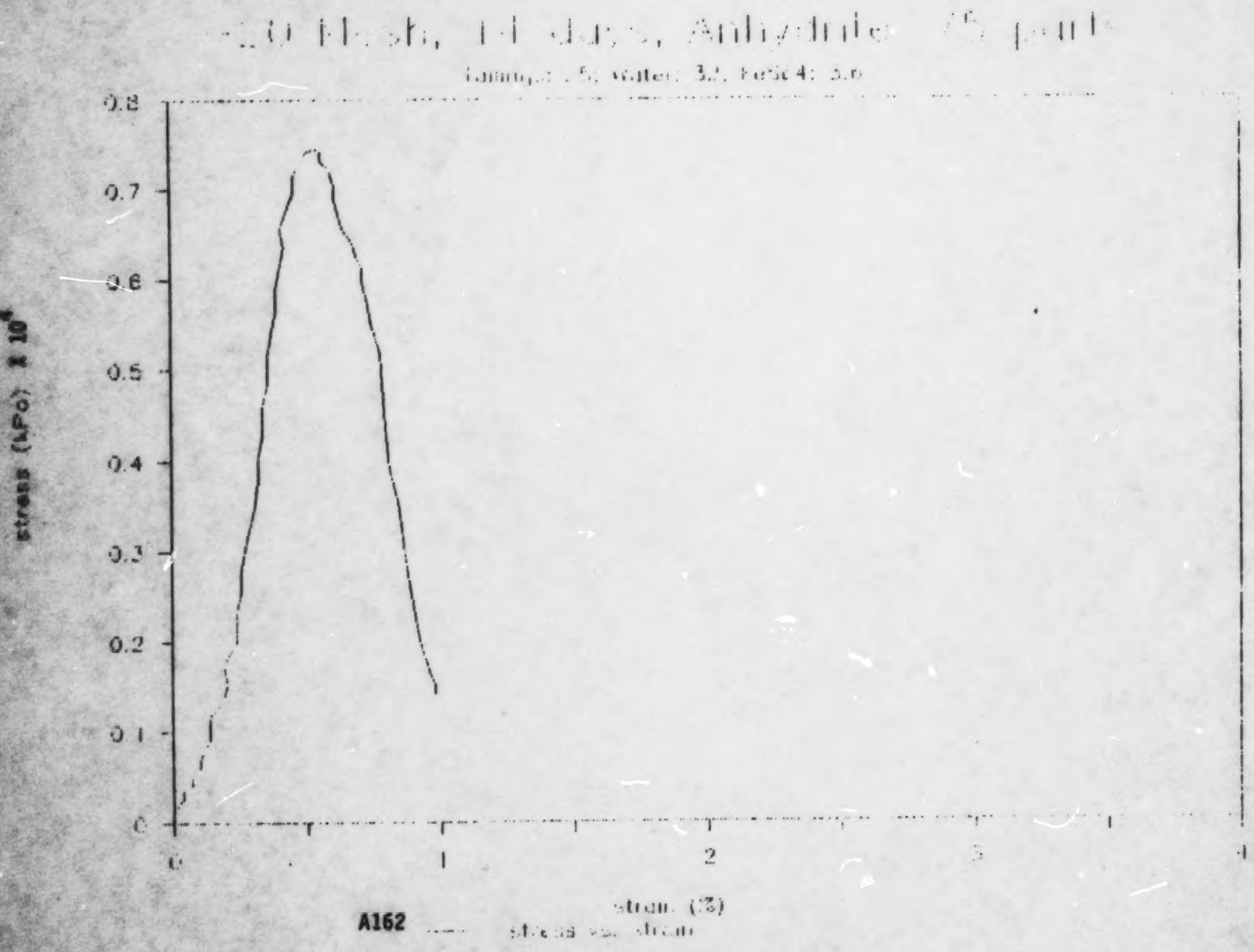

-20 Mesh; 14 days; Anhydrite: 75 parts
Tailings: 25; Water: 42; FeSO4: 2.8
stress (kPa) X 10⁴
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A159
stress vs. strain

−20 Mesh; 14 days; Anhydrite, 50 parts

Tailings: 50; Water: 42; FeSo4: 2.8

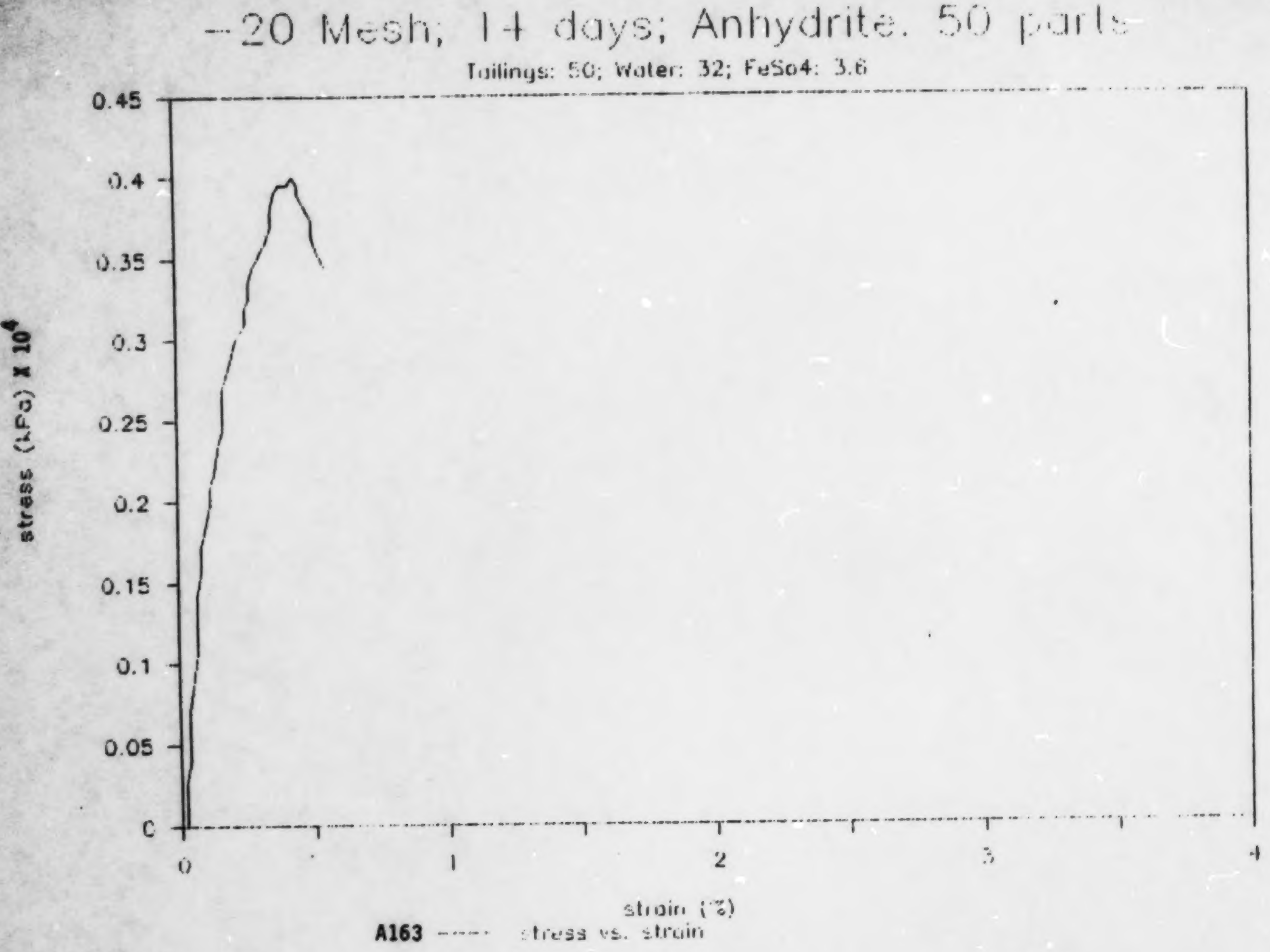



-20 Mesh, 14 days; Anhydrite: 100 parts
tailings: 0, water: 32; FeSO4: 3.6
stress (kPa) X 10⁴
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A161 ------ stress vs. strain



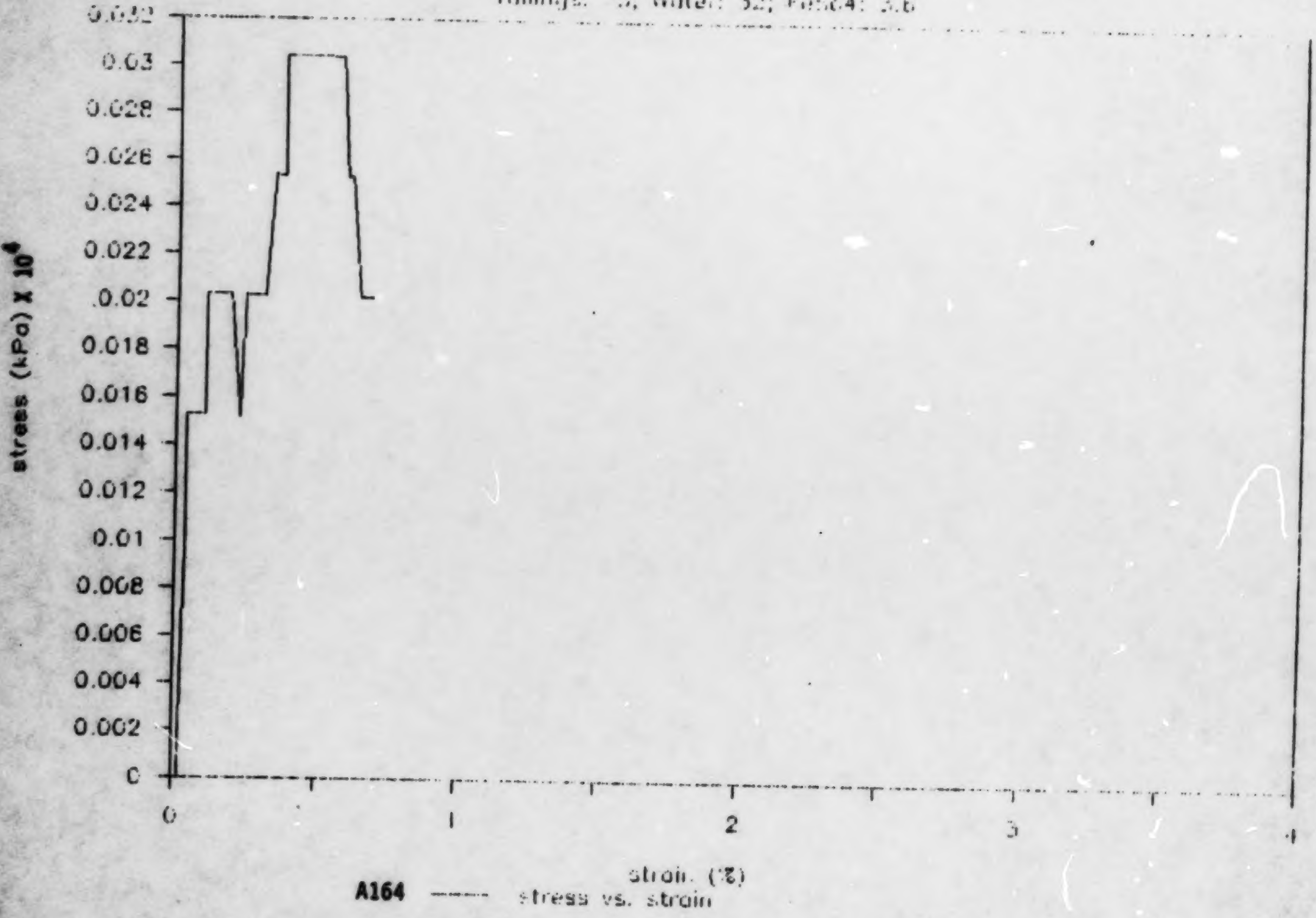
-20 Mesh, 14 days, Anhydrite 75 pcf

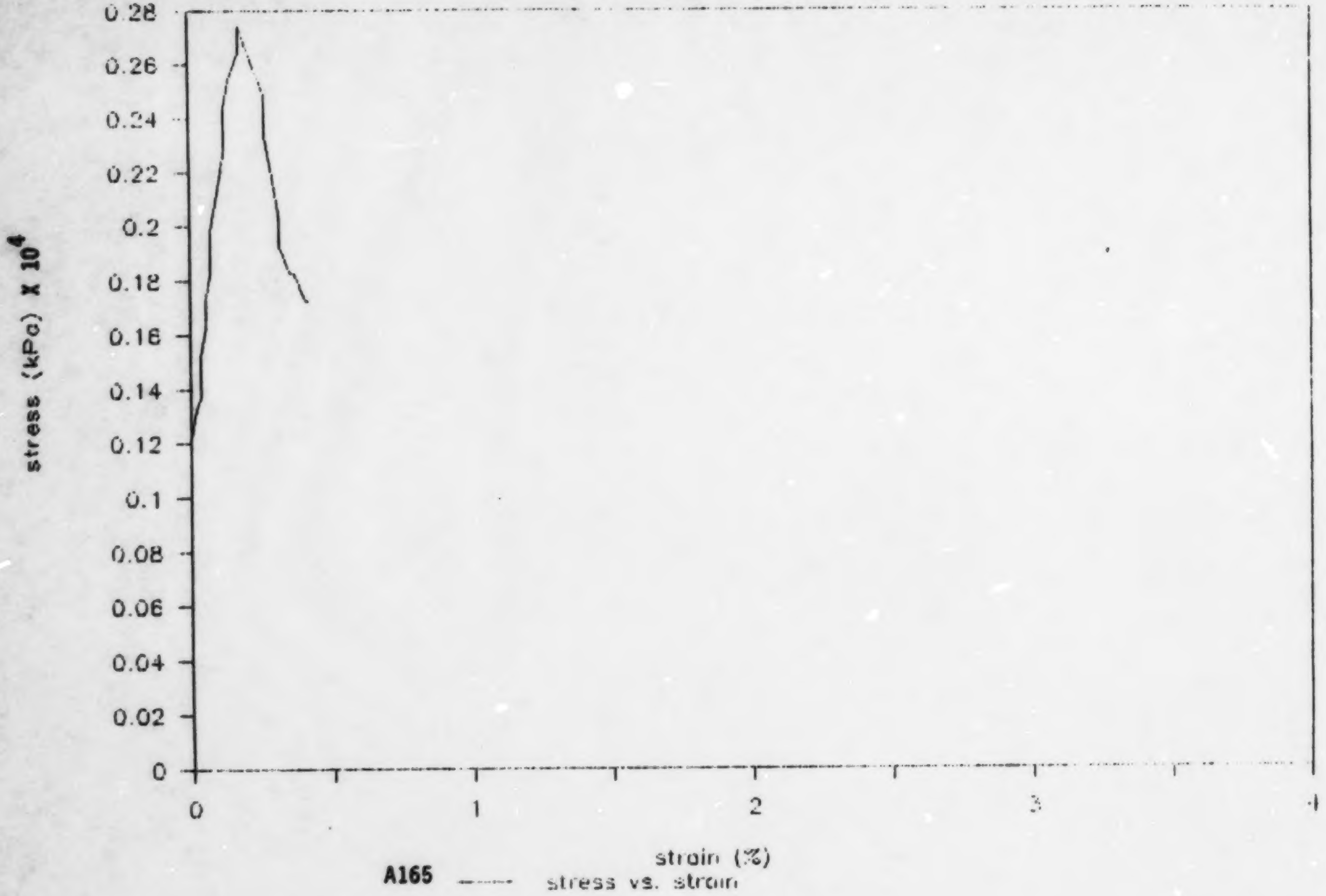
Tailings: 75; Water: 3.2; FeSO4: 5.0


-20 Mesh; 14 days; Anhydrite. 50 parts
Tailings: 50; Water: 32; FeSo4: 3.6
stress (kPa) X 10^4
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
3
4
strain (%)
A163 ----- stress vs. strain

-20 Mesh, 14 days, Anhydrite: 25 pcf

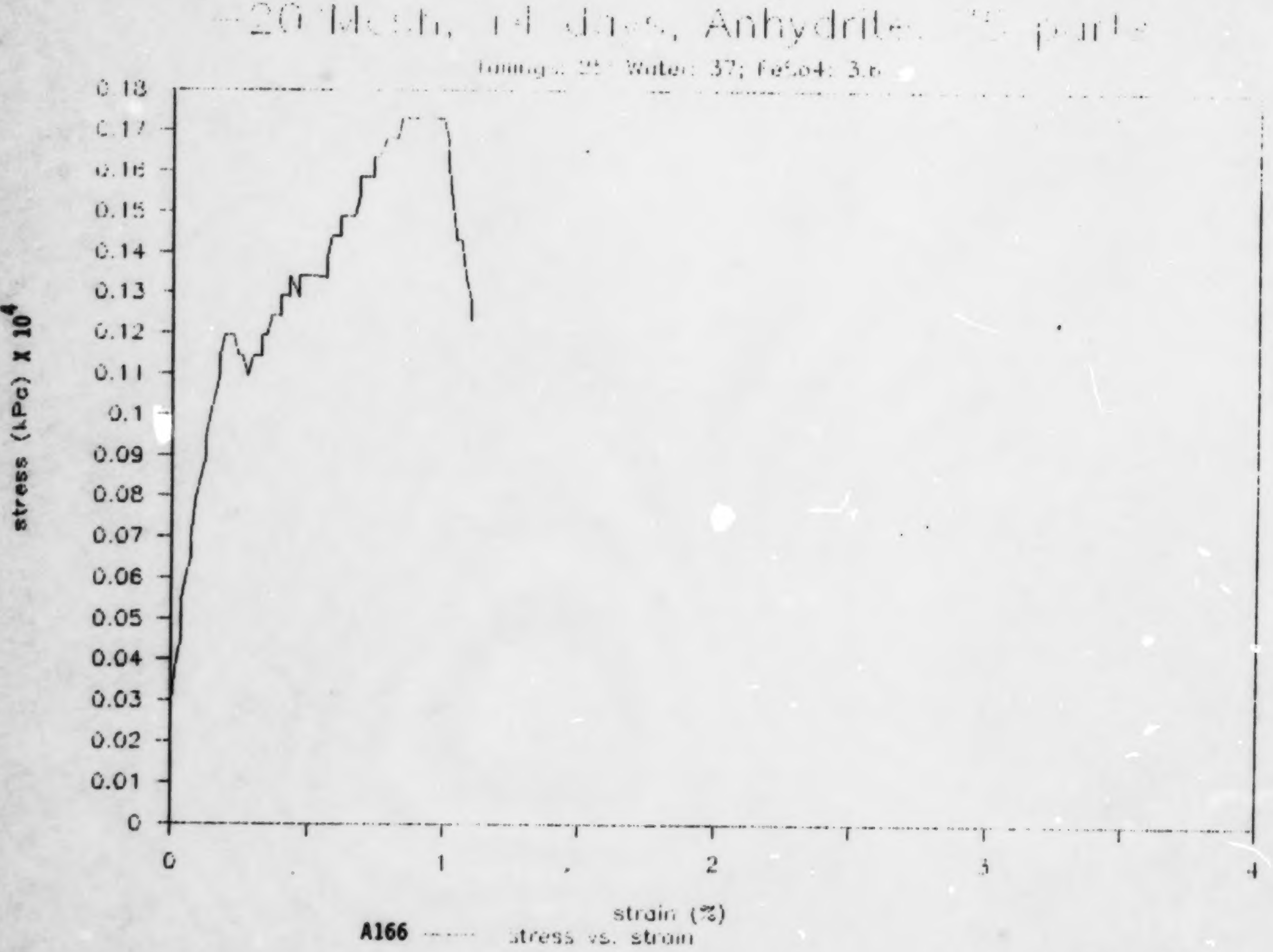
Tailings: 75; Water: 32; FeSO4: 3.6

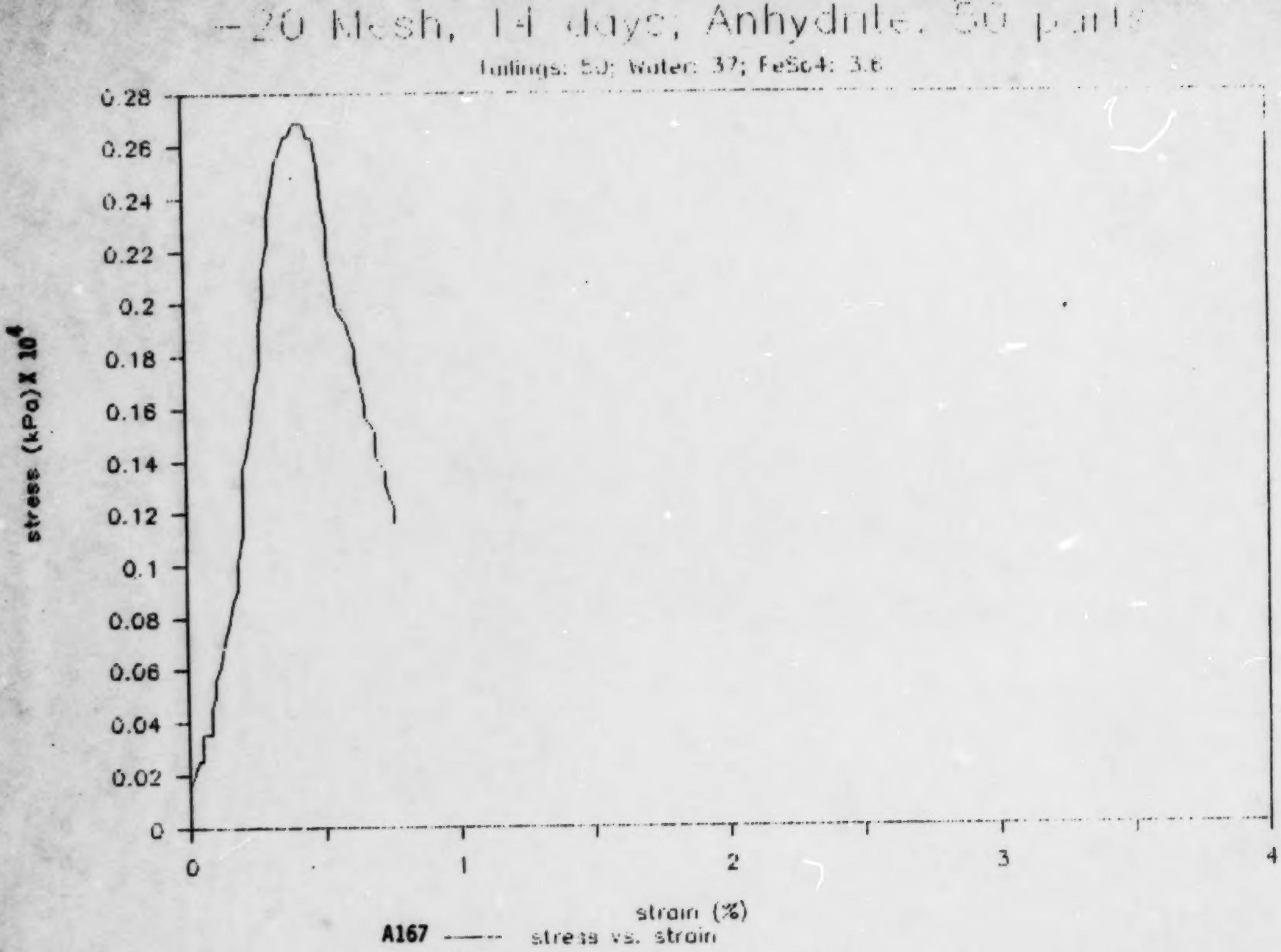


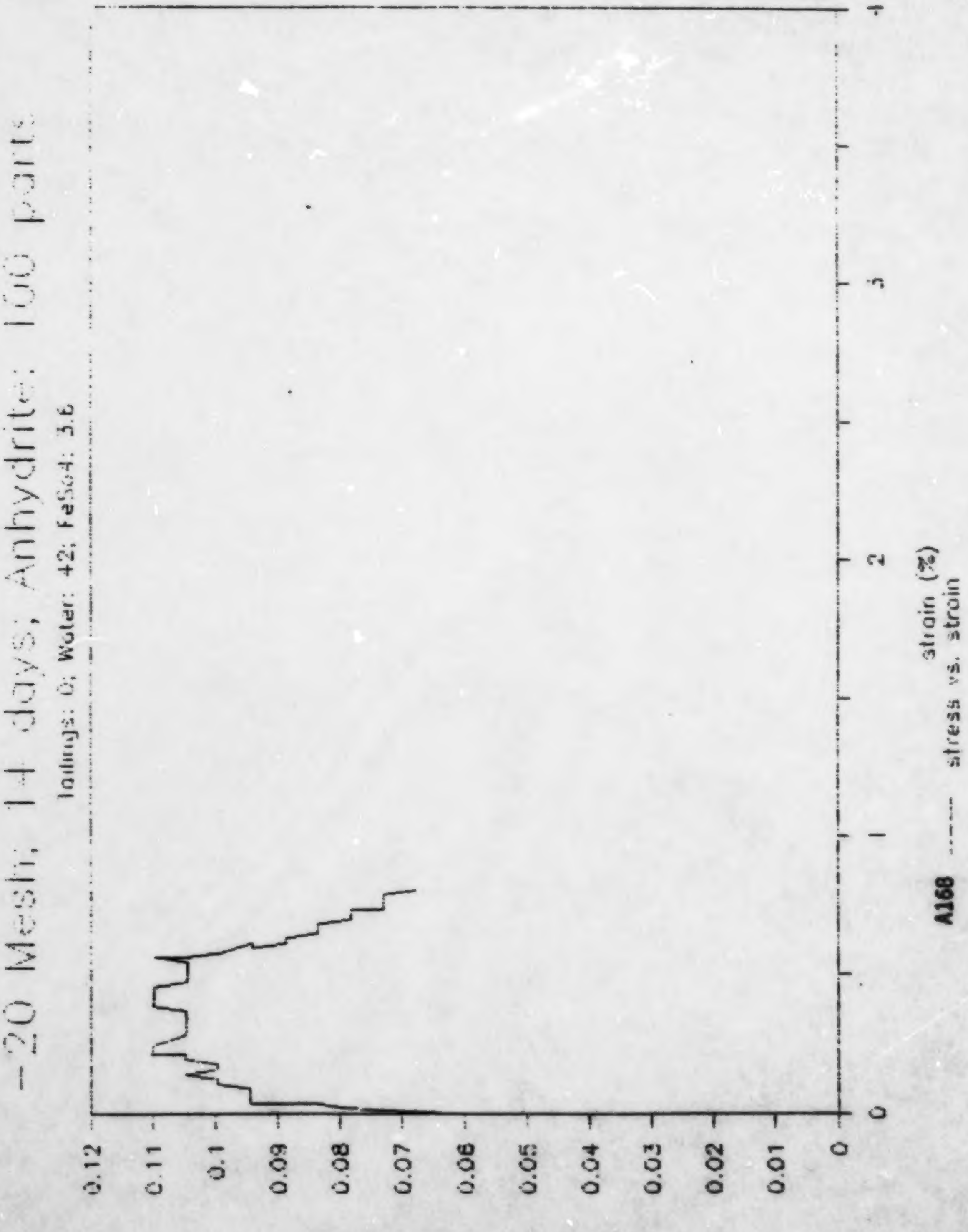


-20 Mesh, 14 days; Anhydrite, 100 parts

Tailings: 0; Water: 57; Feso4: 3.6

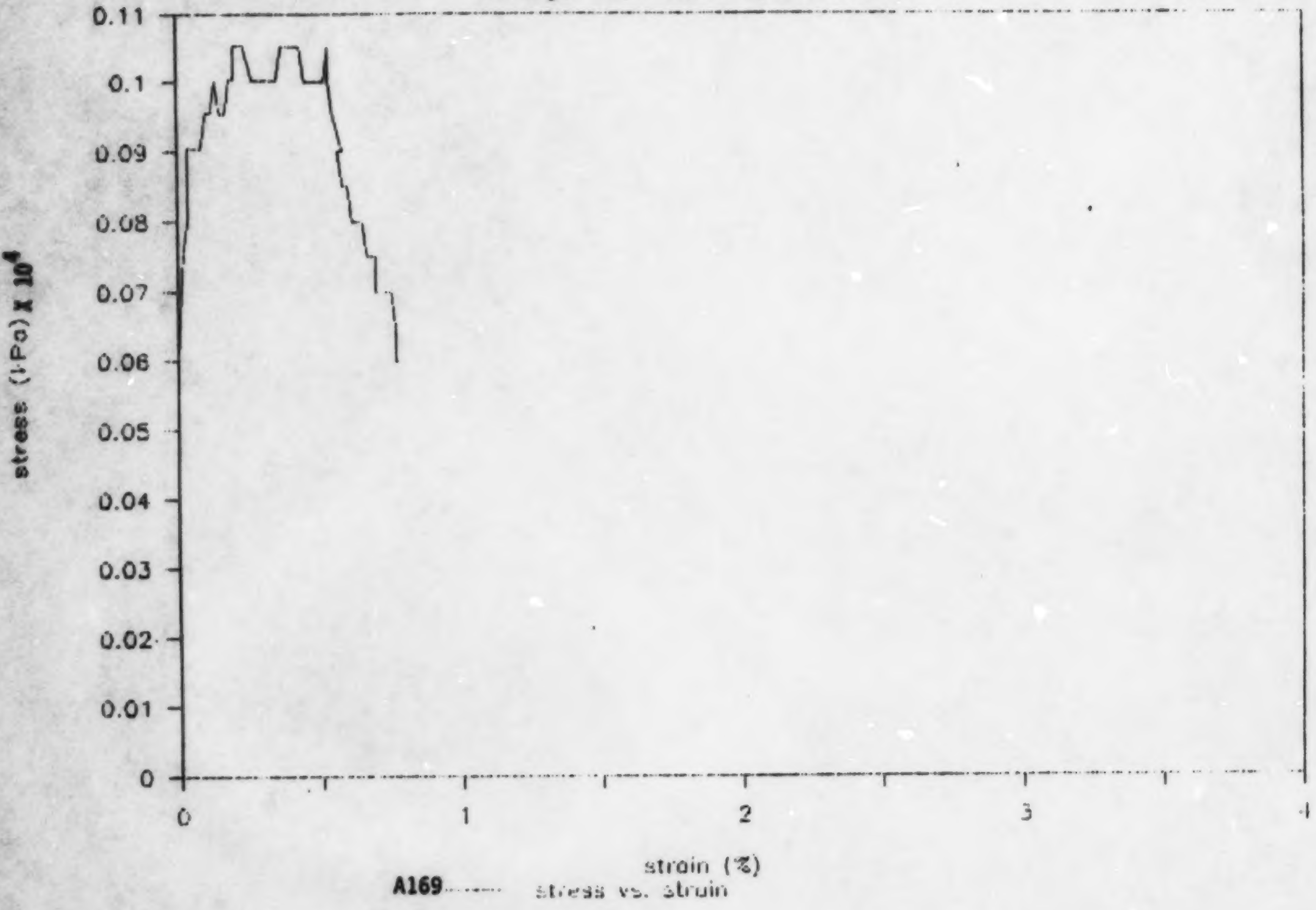

-20 Mesh, 14 days, Anhydrite: [illegible] parts
Tailings: 2[illegible] Water: 37; FeSo4: 3.6
stress (kPa) X 10^4
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A166
stress vs. strain


-20 Mesh, 14 days; Anhydrite, 50 parts
Tailings: 50; Water: 37; FeSo4: 3.6
stress (kPa) X 10⁴
0.28
0.26
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
1
2
3
4
strain (%)
A167 —— stress vs. strain


-20 Mesh; 14 days; Anhydrite: 100 parts
Tailings: 0; Water: 42; FeSO4: 3.6
stress (kPa) X 10⁴
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A168 ----- stress vs. strain

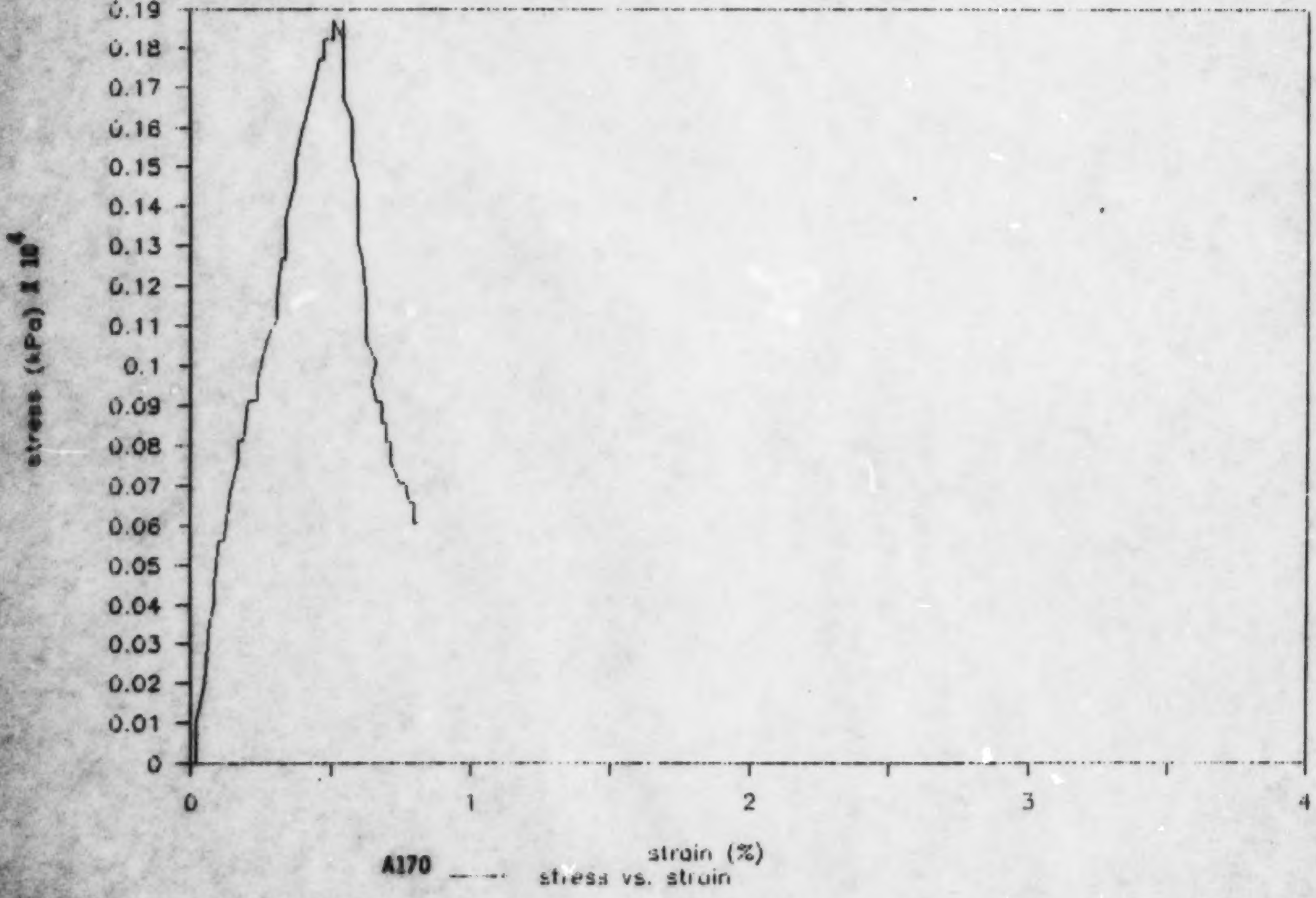
–20 Mesh, 14 days; Anhydrite. 75 parts

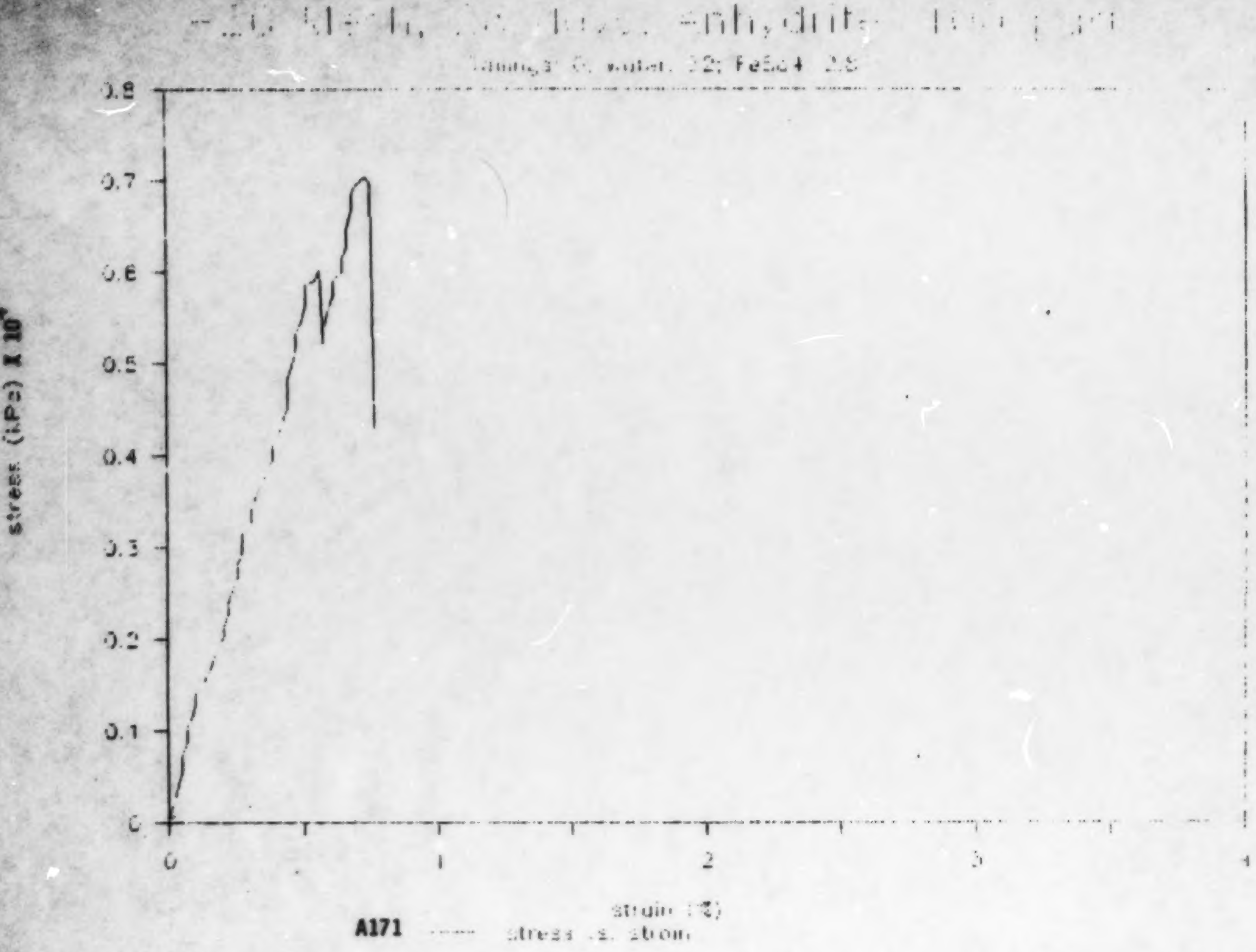
Tailings: 25; Water: 42; FeSo4: 3.6



-20 Mesh; 14 days; Anhydrite: 50 parts

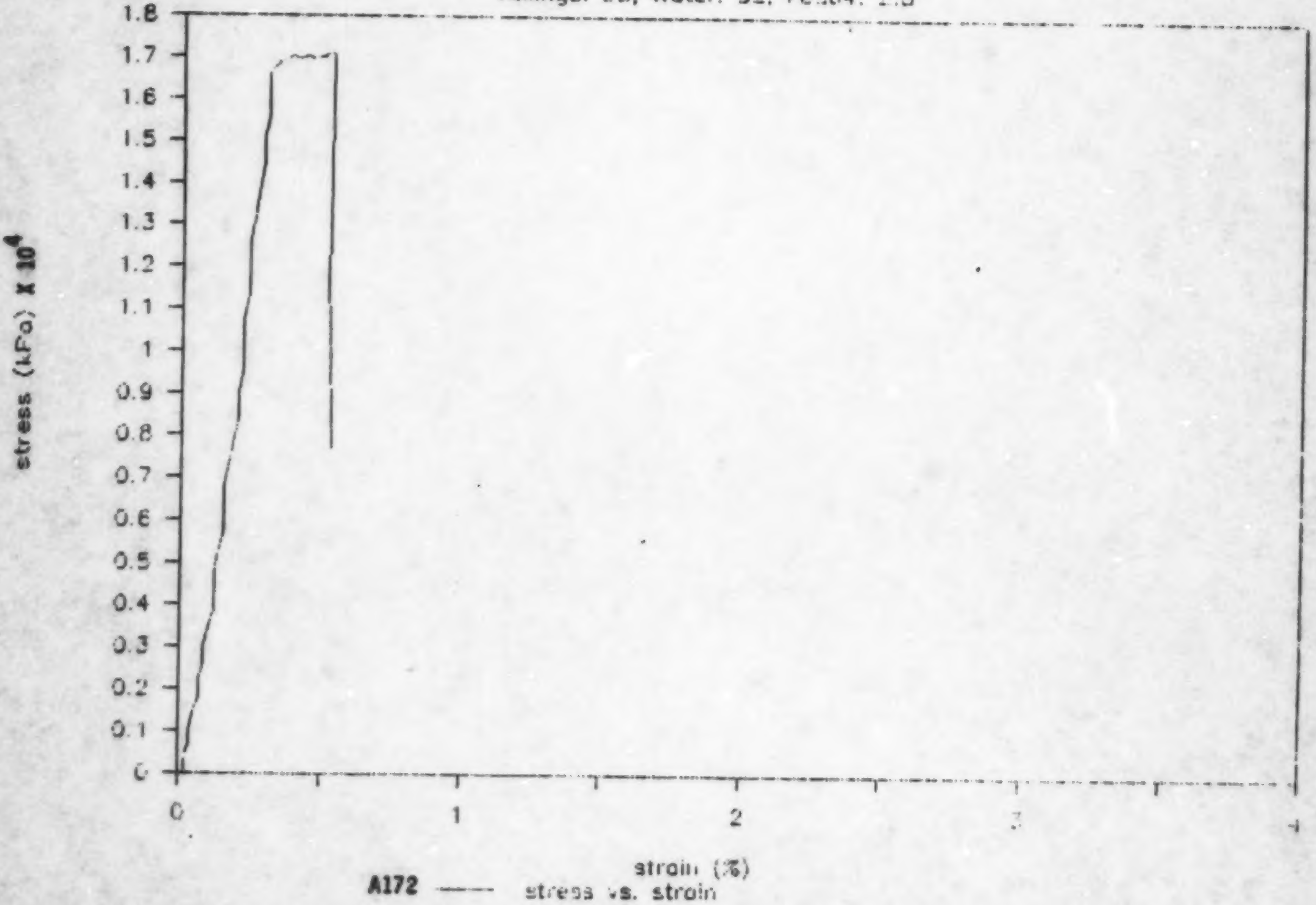
Tailings: 50; Water: 42; FeSO4: 3.6

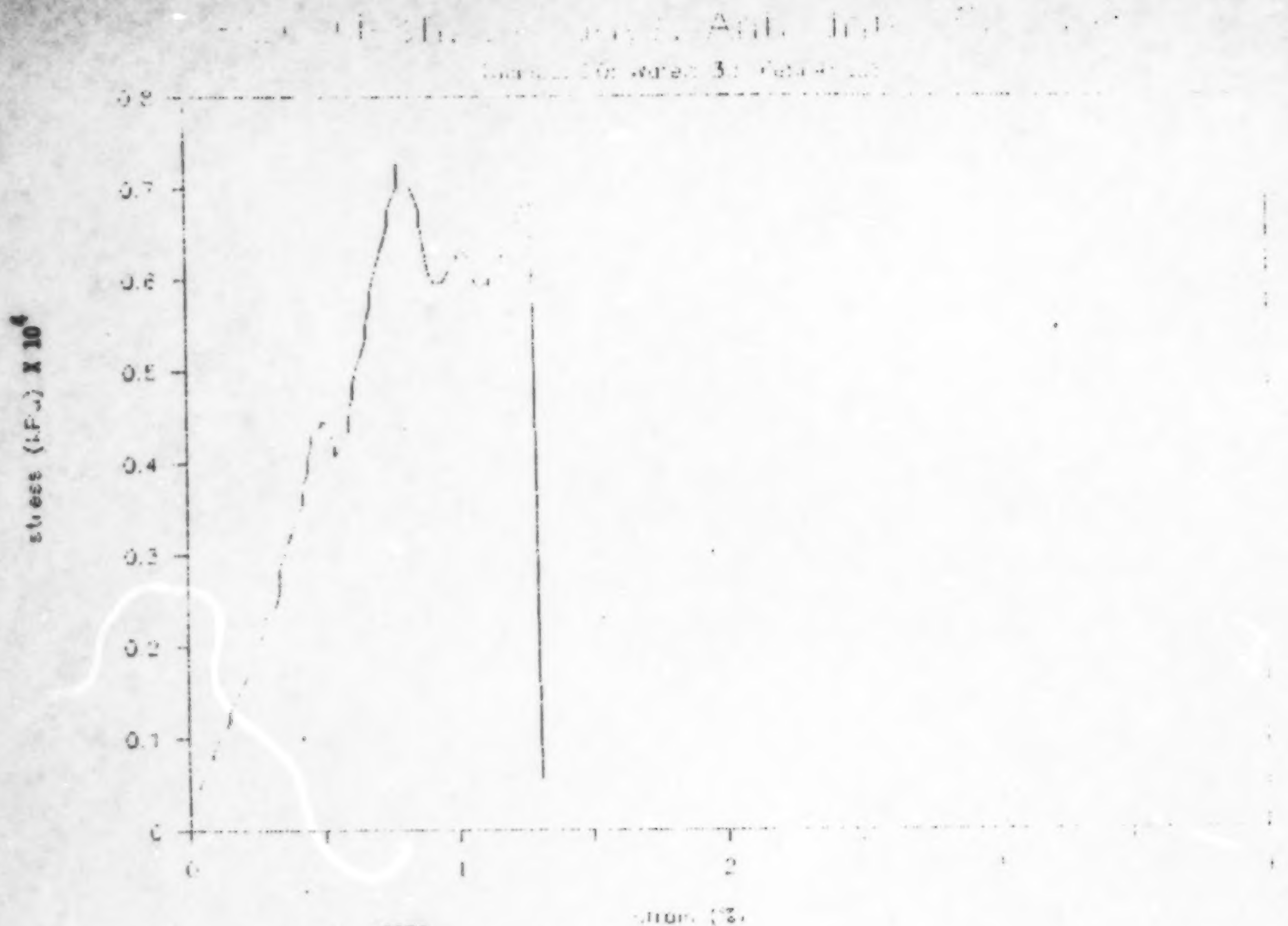



[illegible] anhydrite [illegible]
[illegible] water: 32; FeSO4 2.8
stress (kPa) X 10^4
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
strain (%)
A171
stress vs. strain

-20 Mesh; 28 days; Anhydrite 75 parts

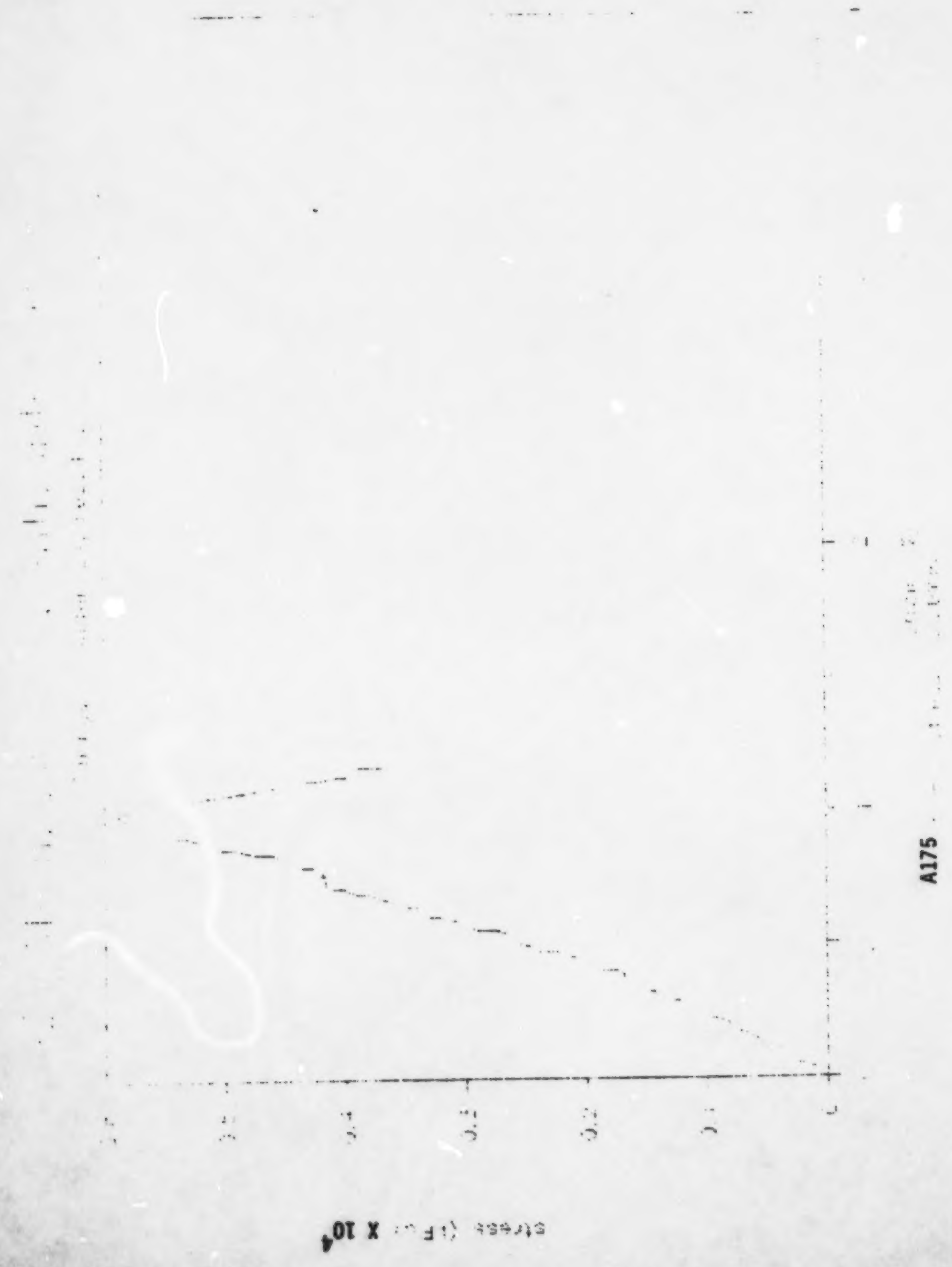
Tailings: 25; Water: 32; Feso4: 2.8

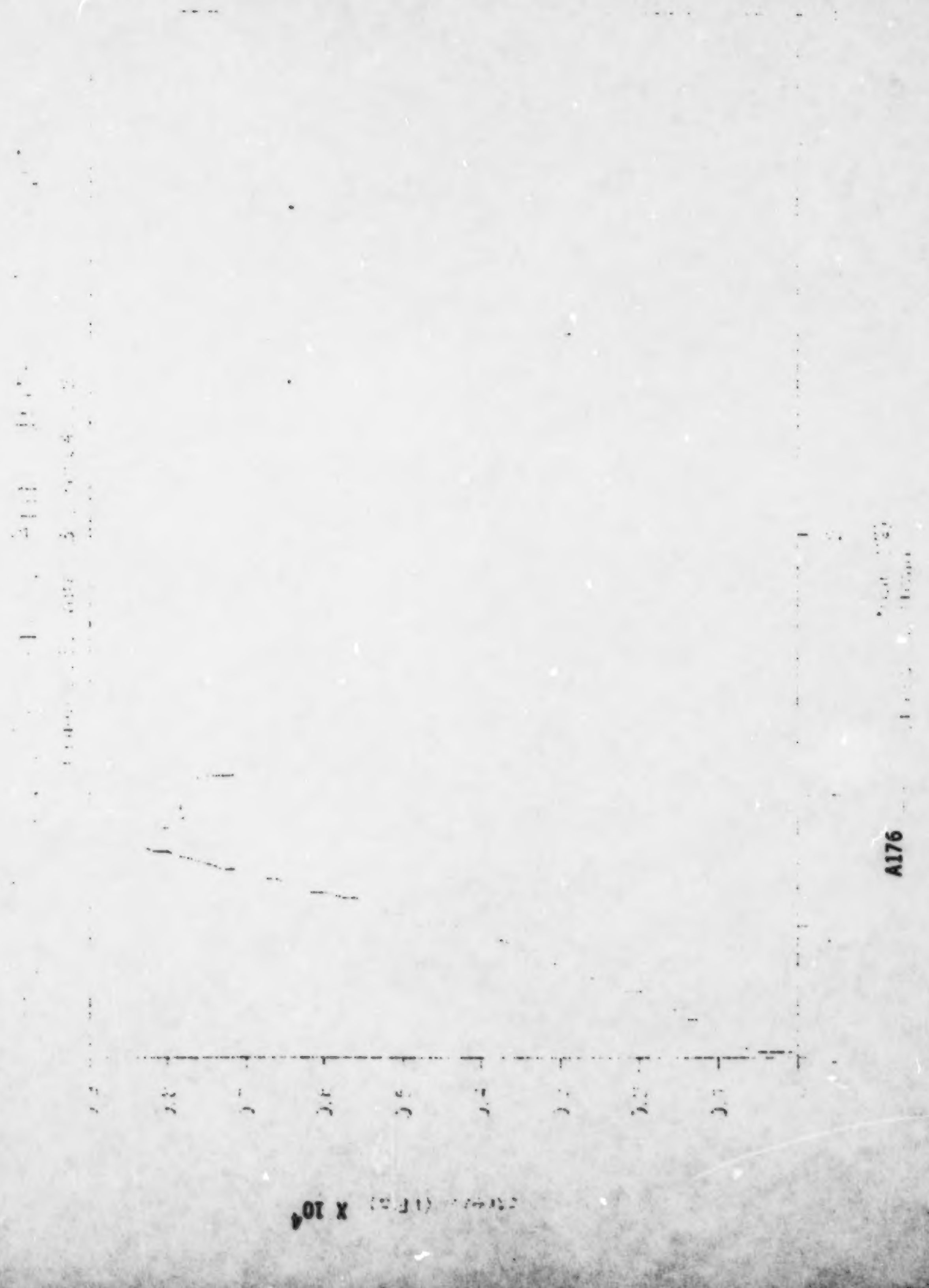




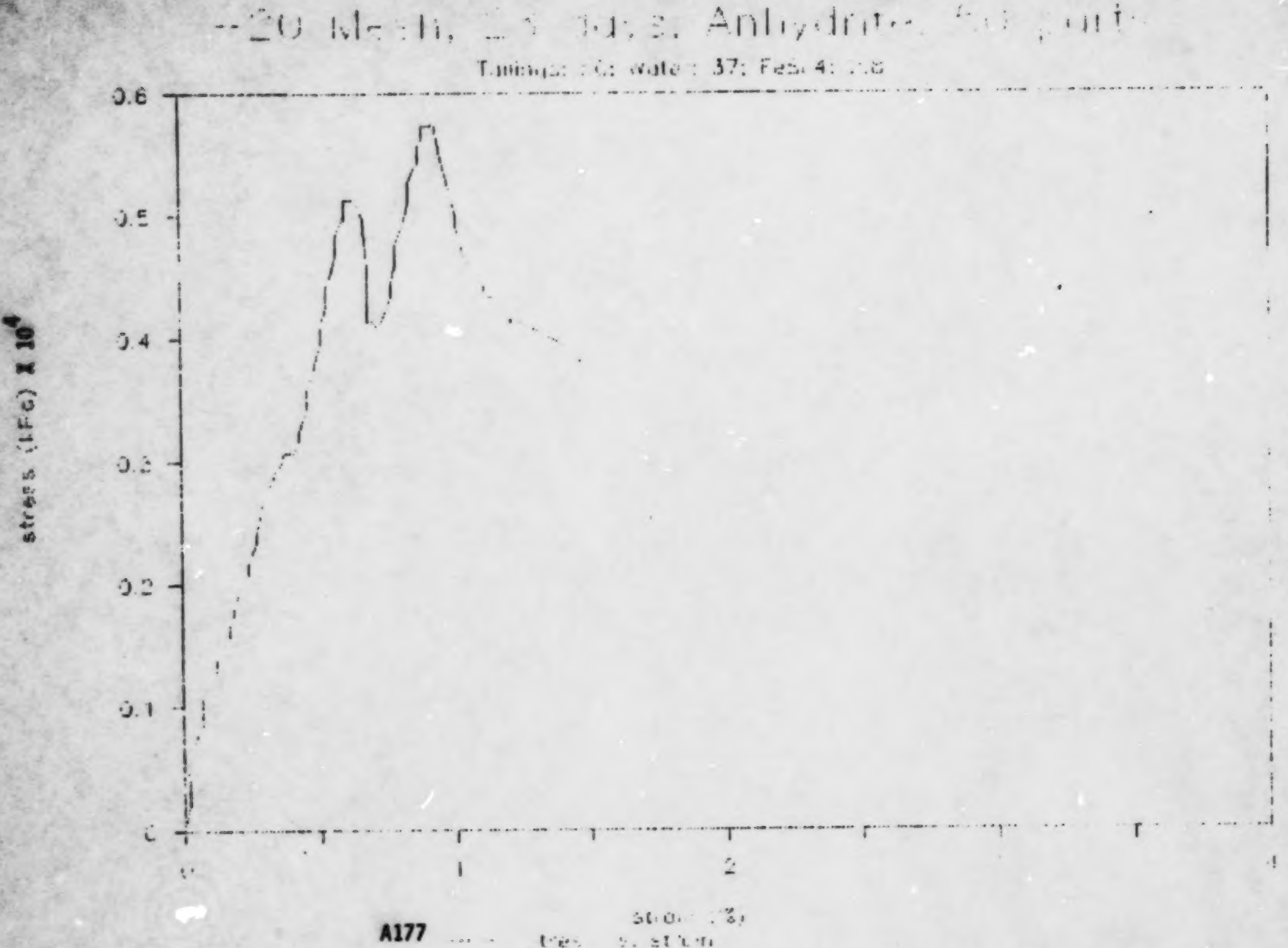
A173 —— stress vs. strain

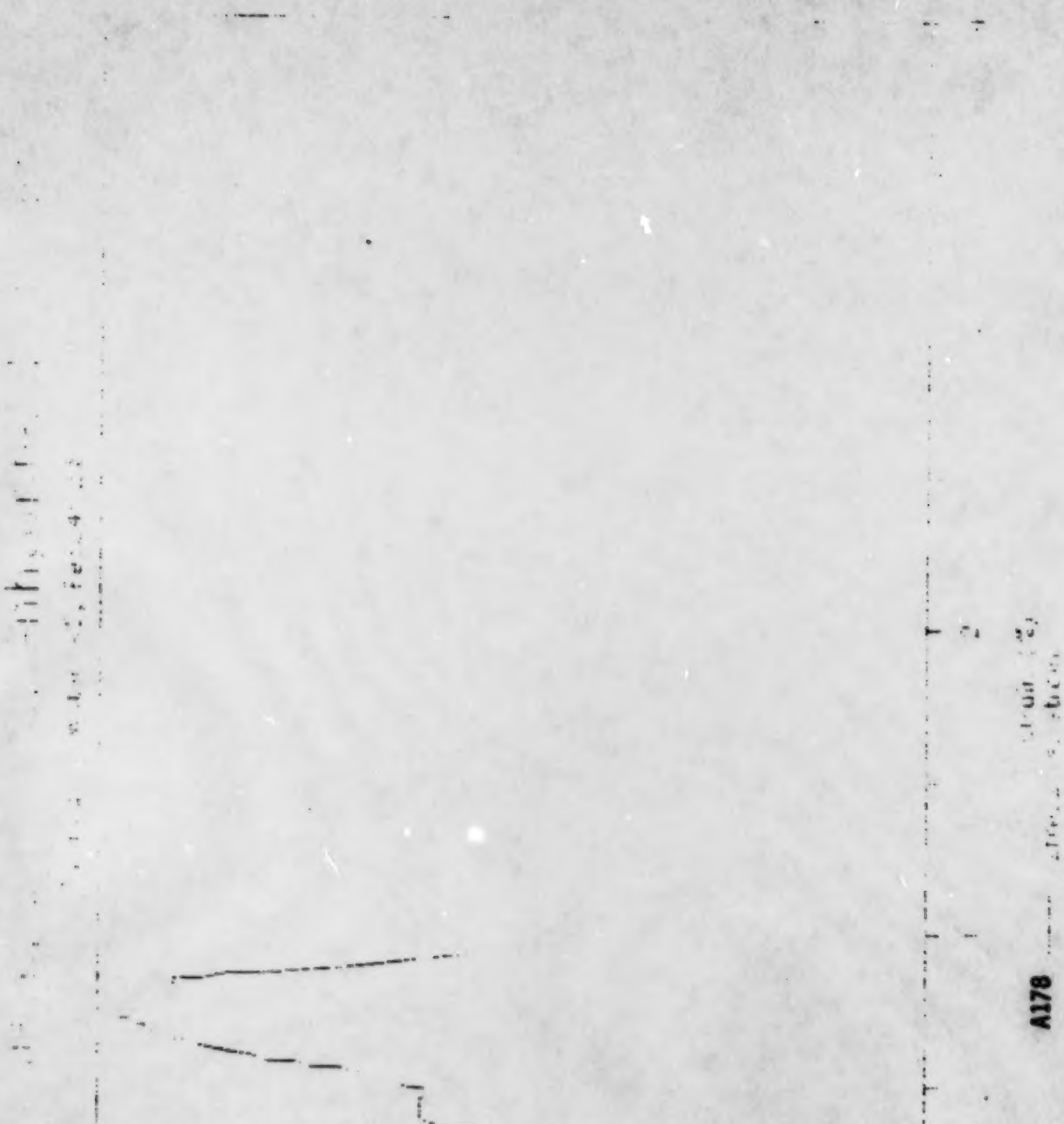
A174

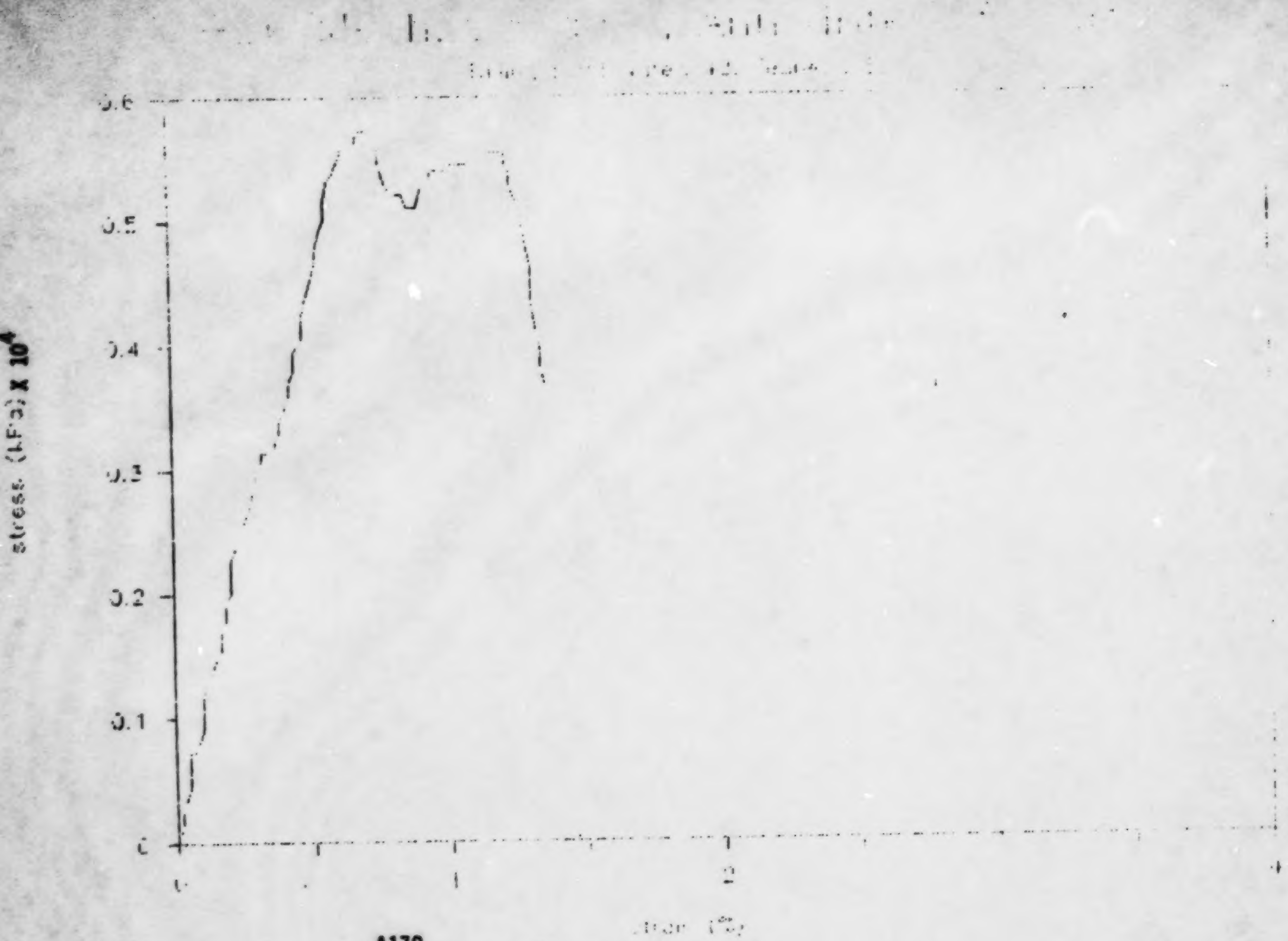

A175
X 10^4
0



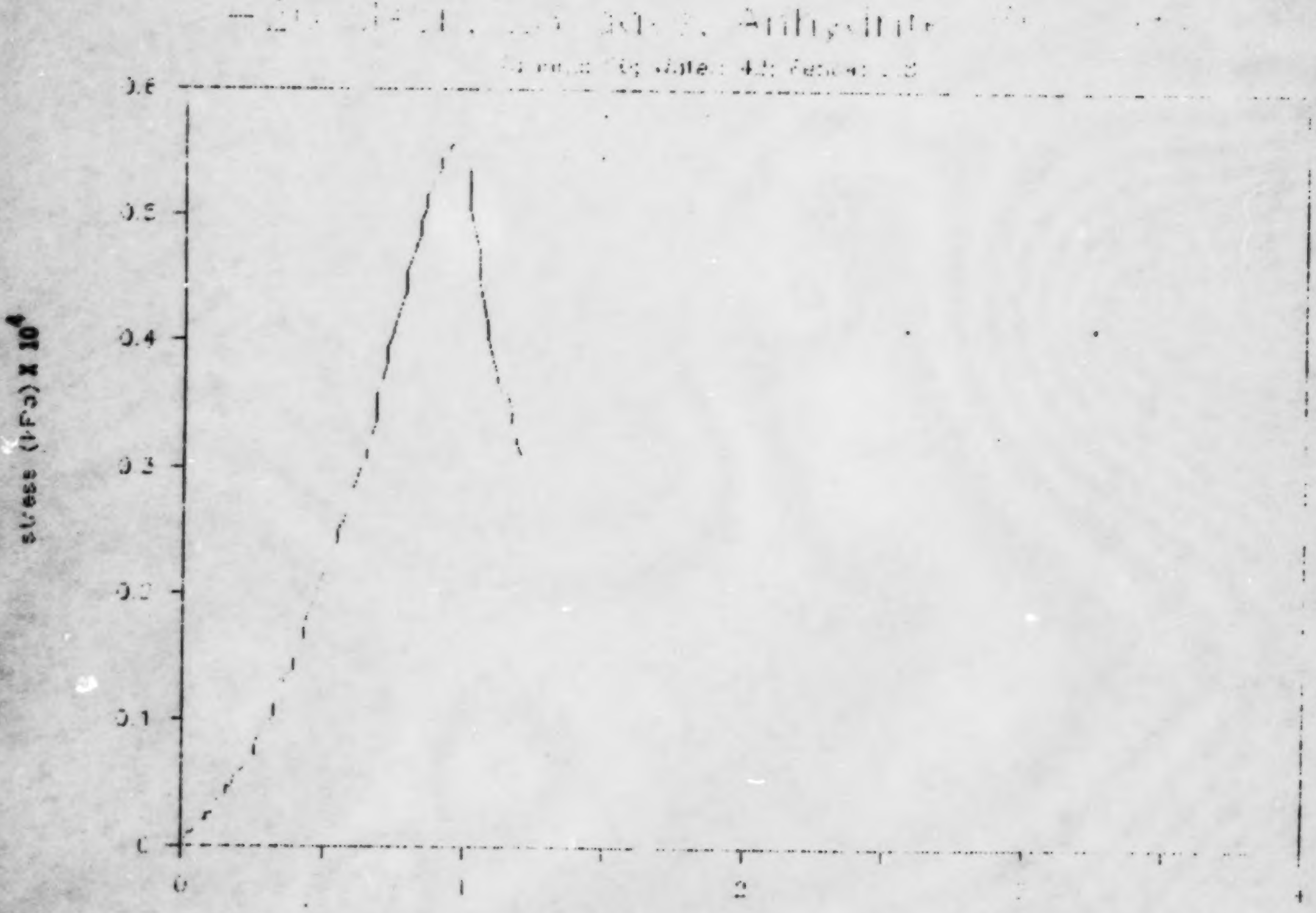
A176


-20 Mesh, [illegible] Anhydrite [illegible]
Tamping: [illegible]; Water: 37; Fes: 4: [illegible]
stress (LFc) X 10^4
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
4
strain (%)
A177
stress vs. strain

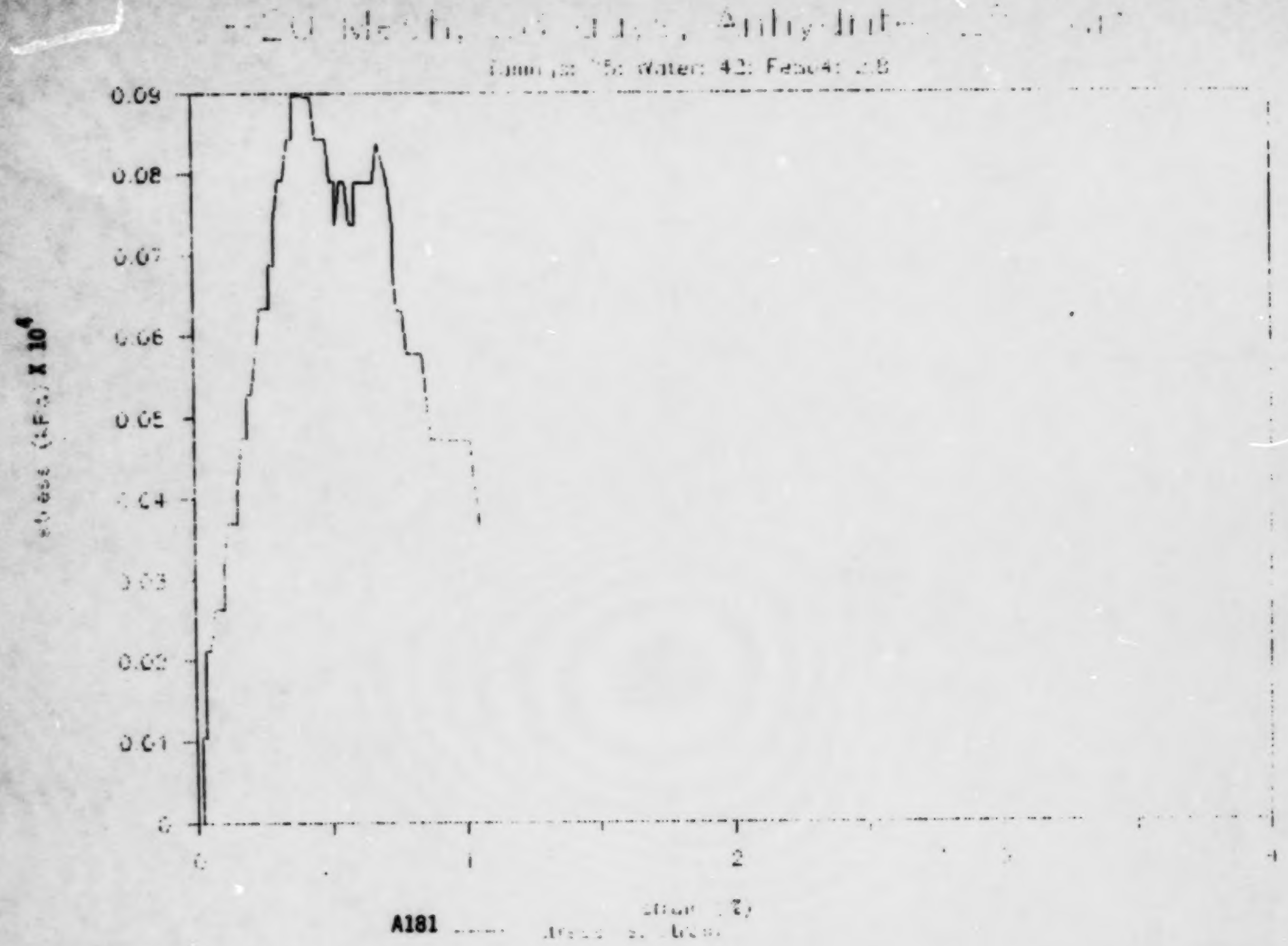

Stress (lb/in²) X 10⁴
0.18
0.16
0.14
0.12
0.06
0.04
0.02
0
Strain (%)
A178

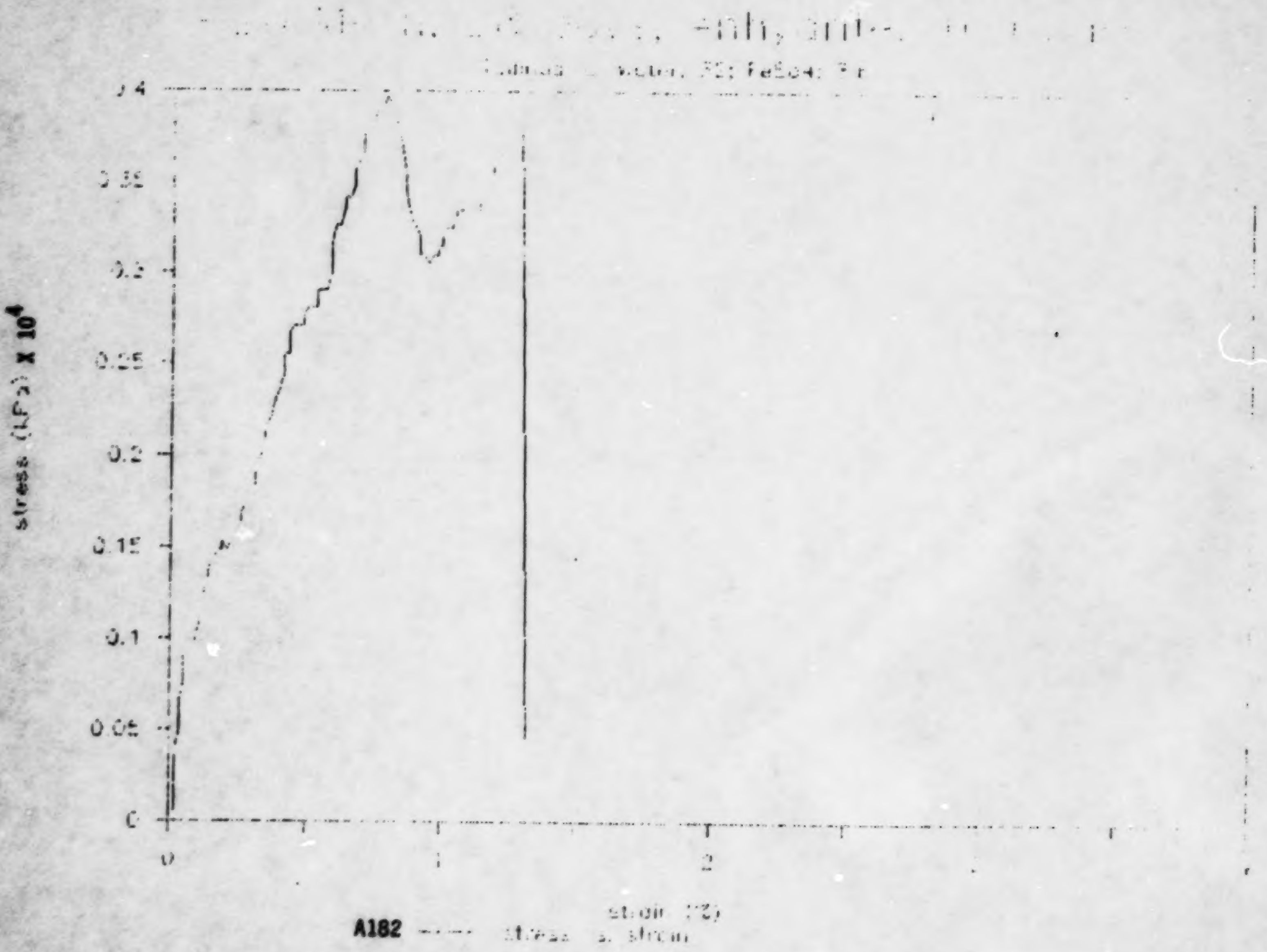


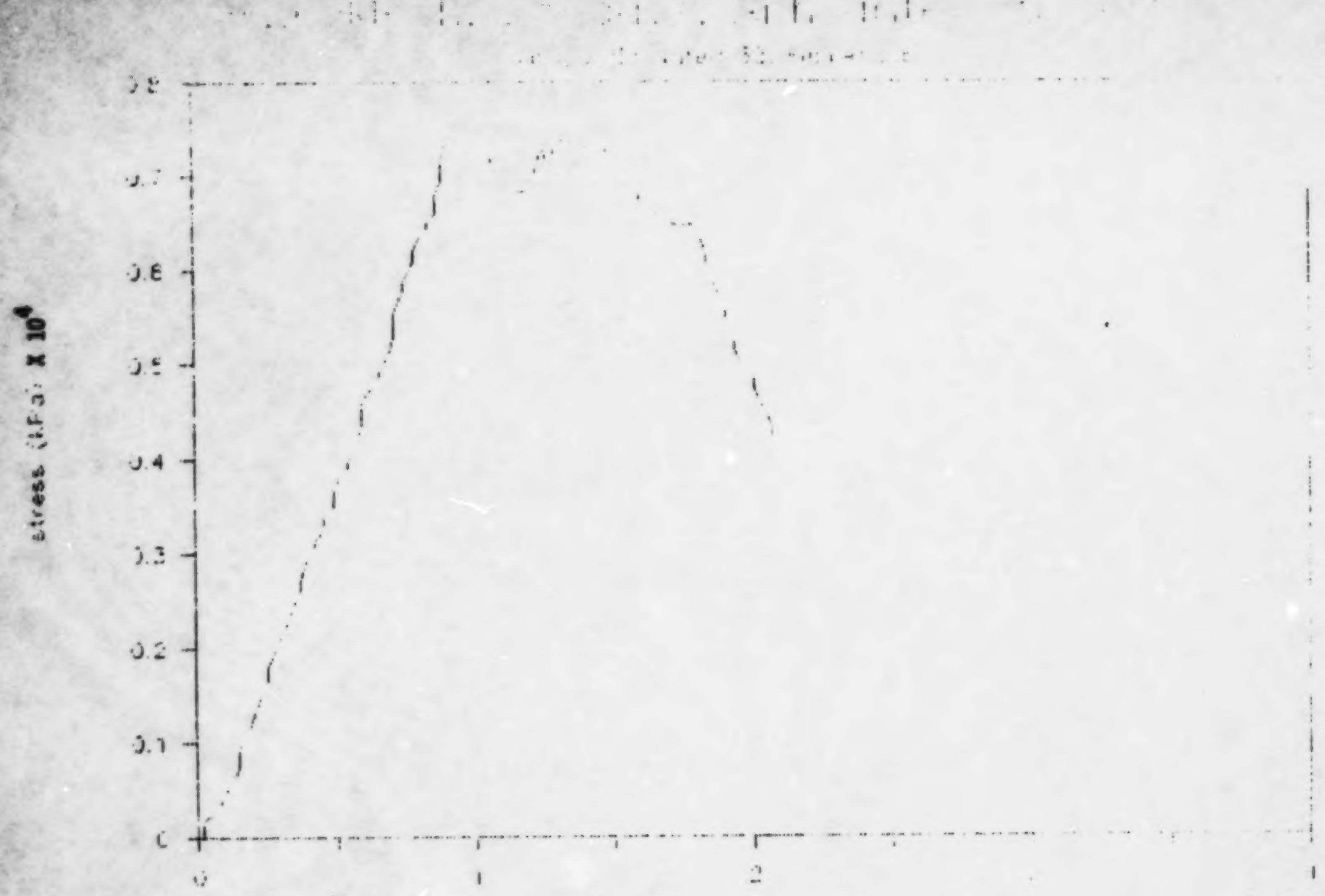
A179



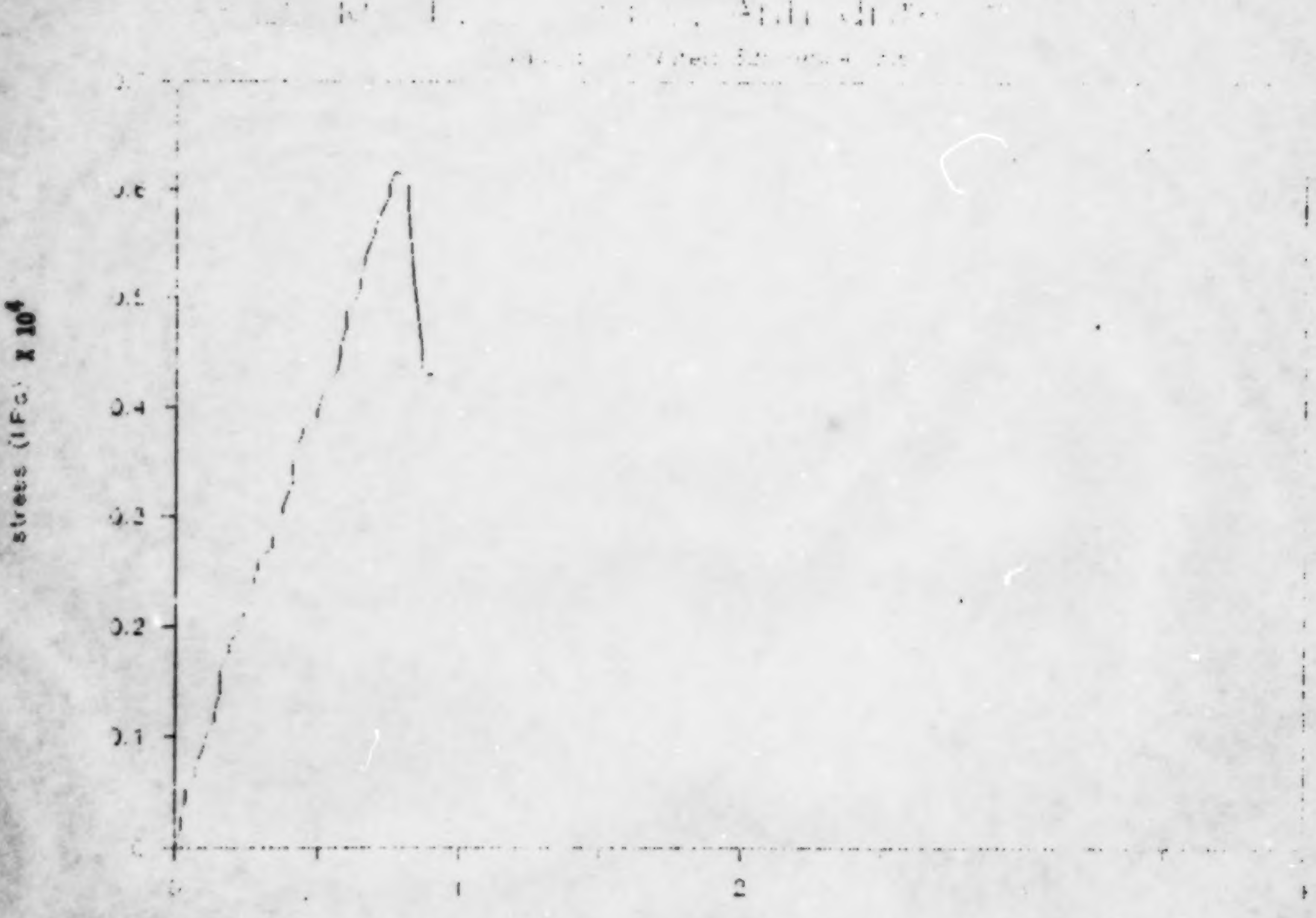
A180 —— stress vs. strain


Anhydrite
Water: 42; FeSO4
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
(KPa) X 10^4
0
1
2
3
4
Strain (%)
A181

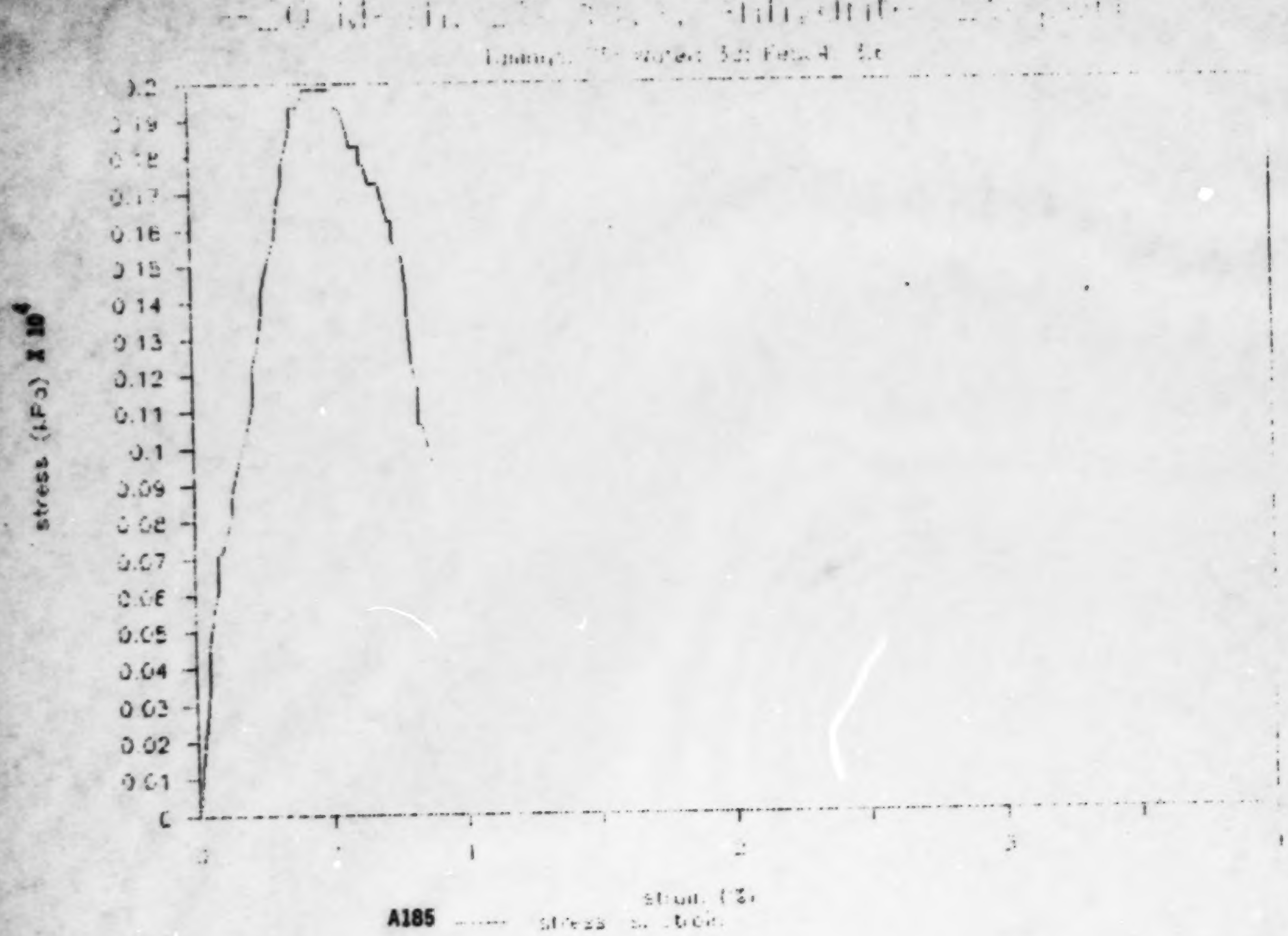

stress (kPa) X 10^4
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
1
2
strain (%)
A182 —·— stress vs. strain

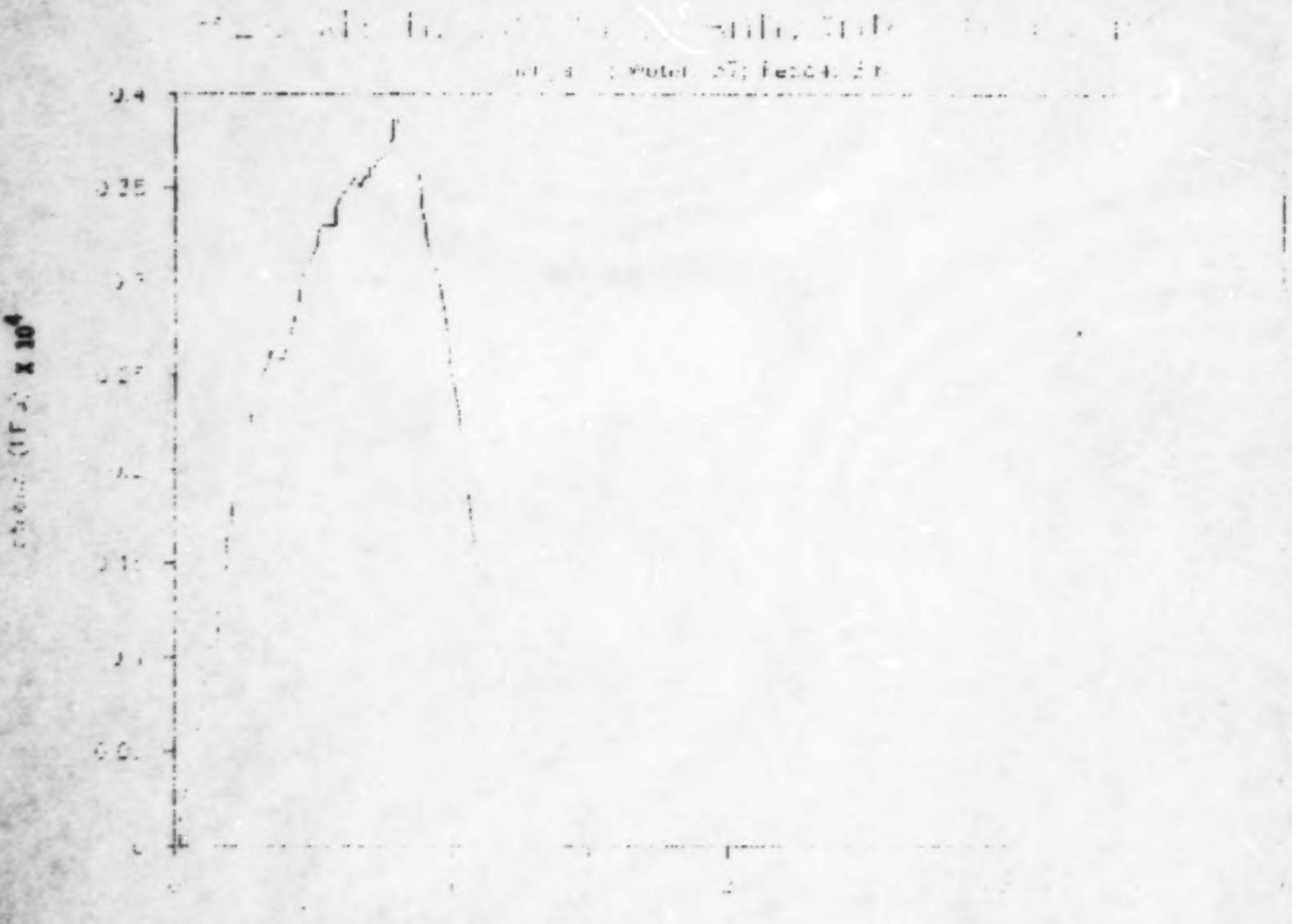


A183 ...... stress vs. strain



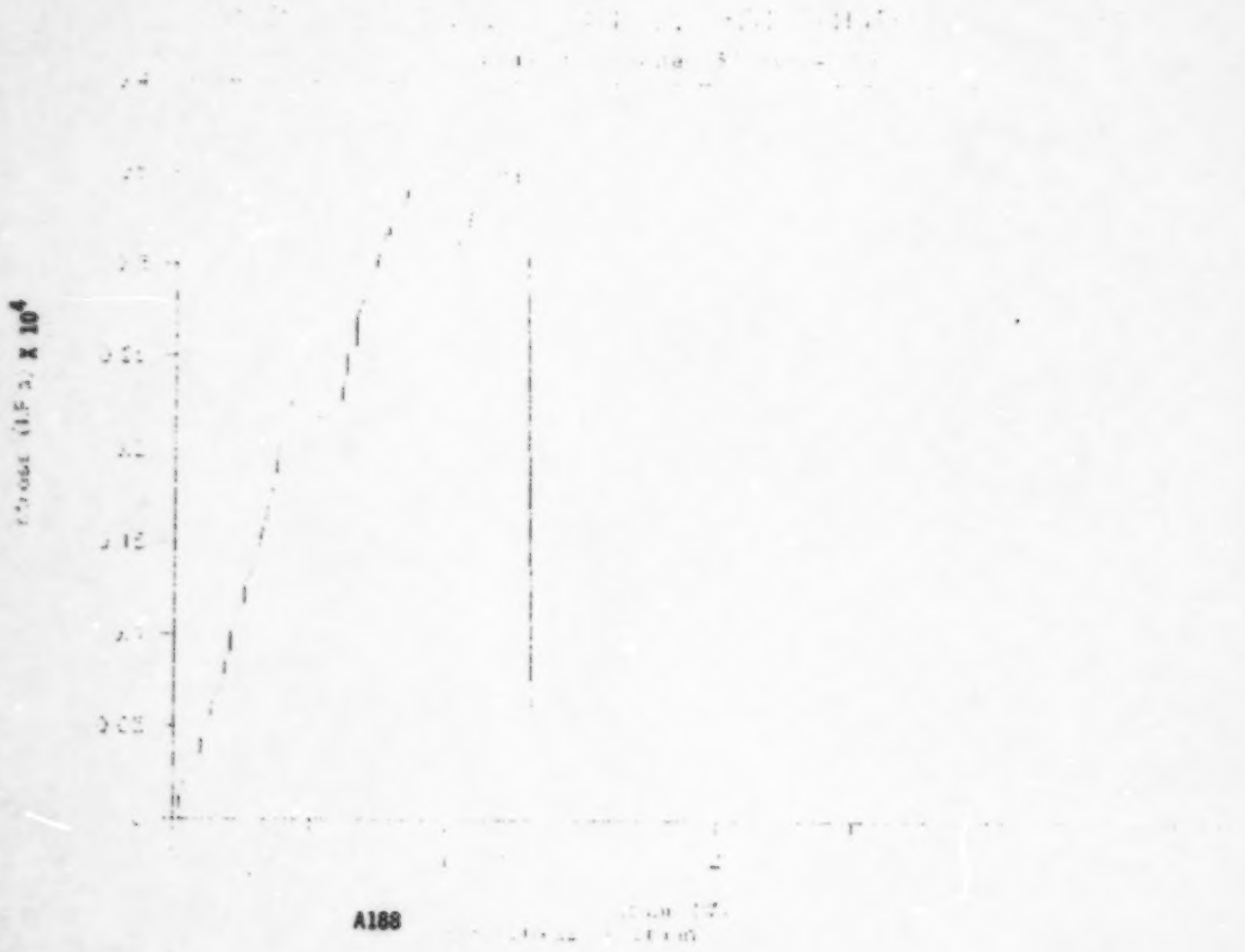
A184


stress (MPa) X 10⁴
0.2
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0
1
2
3
4
strain (%)
A185
stress vs. strain

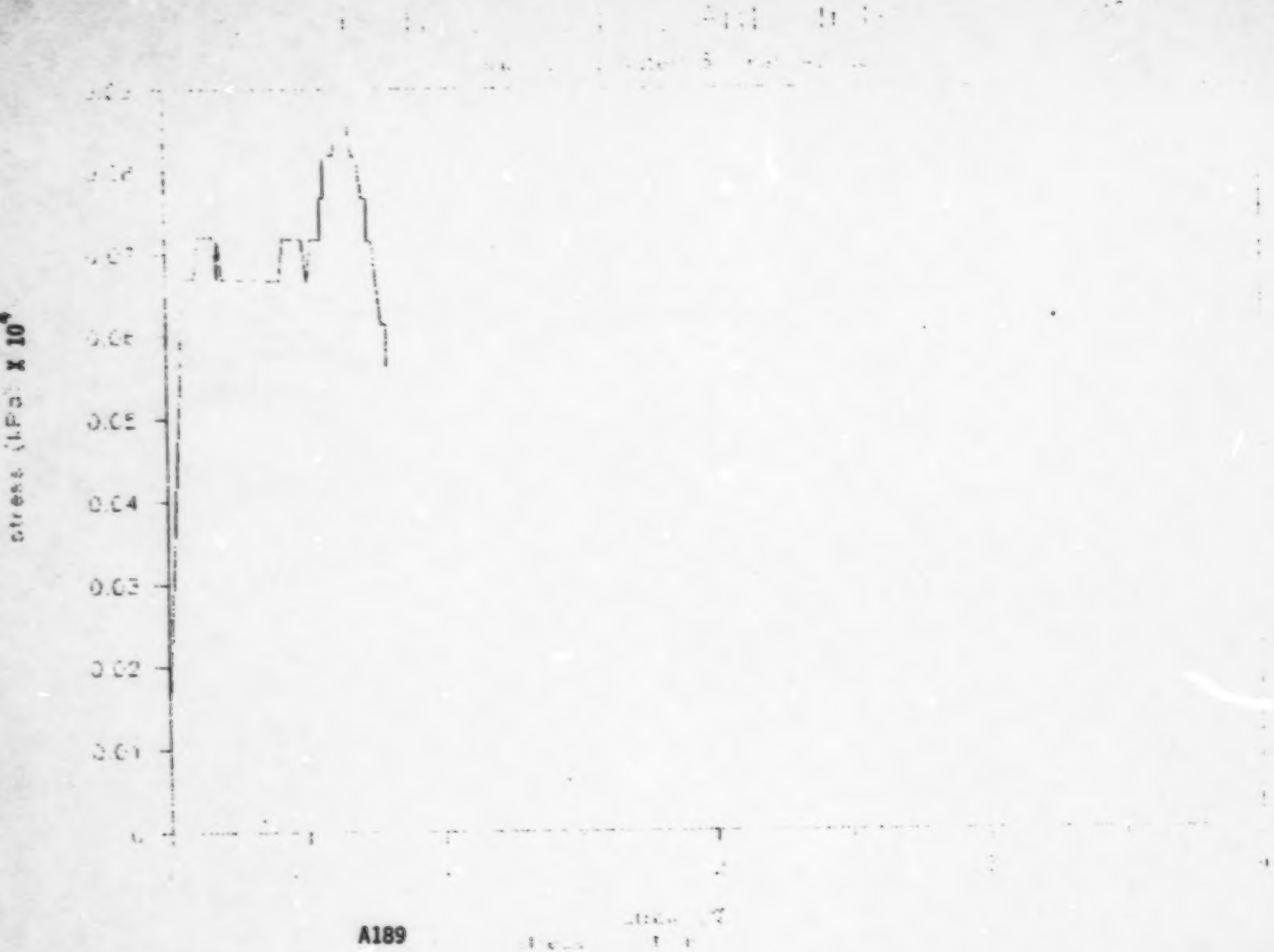


A186

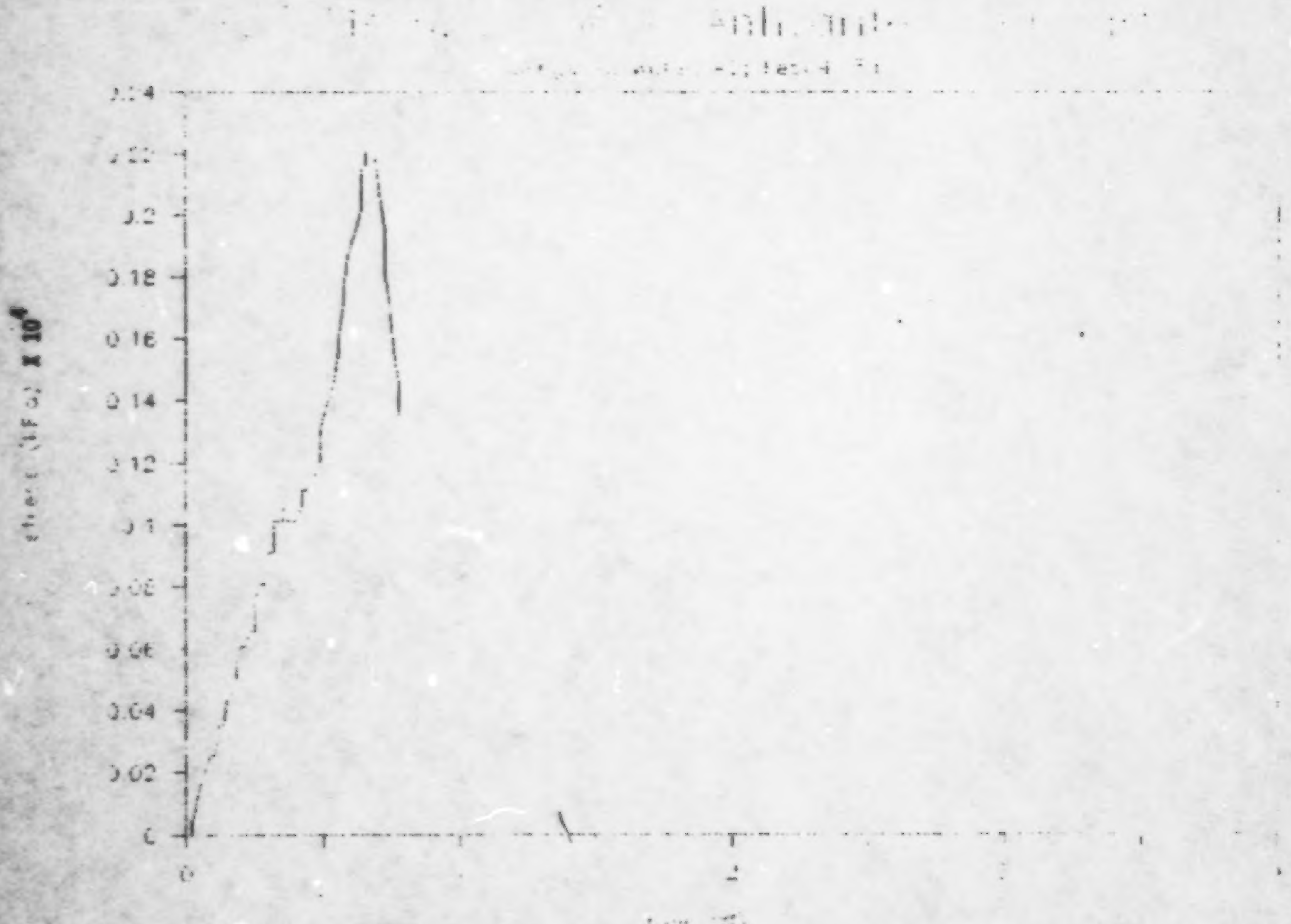
A187



A188



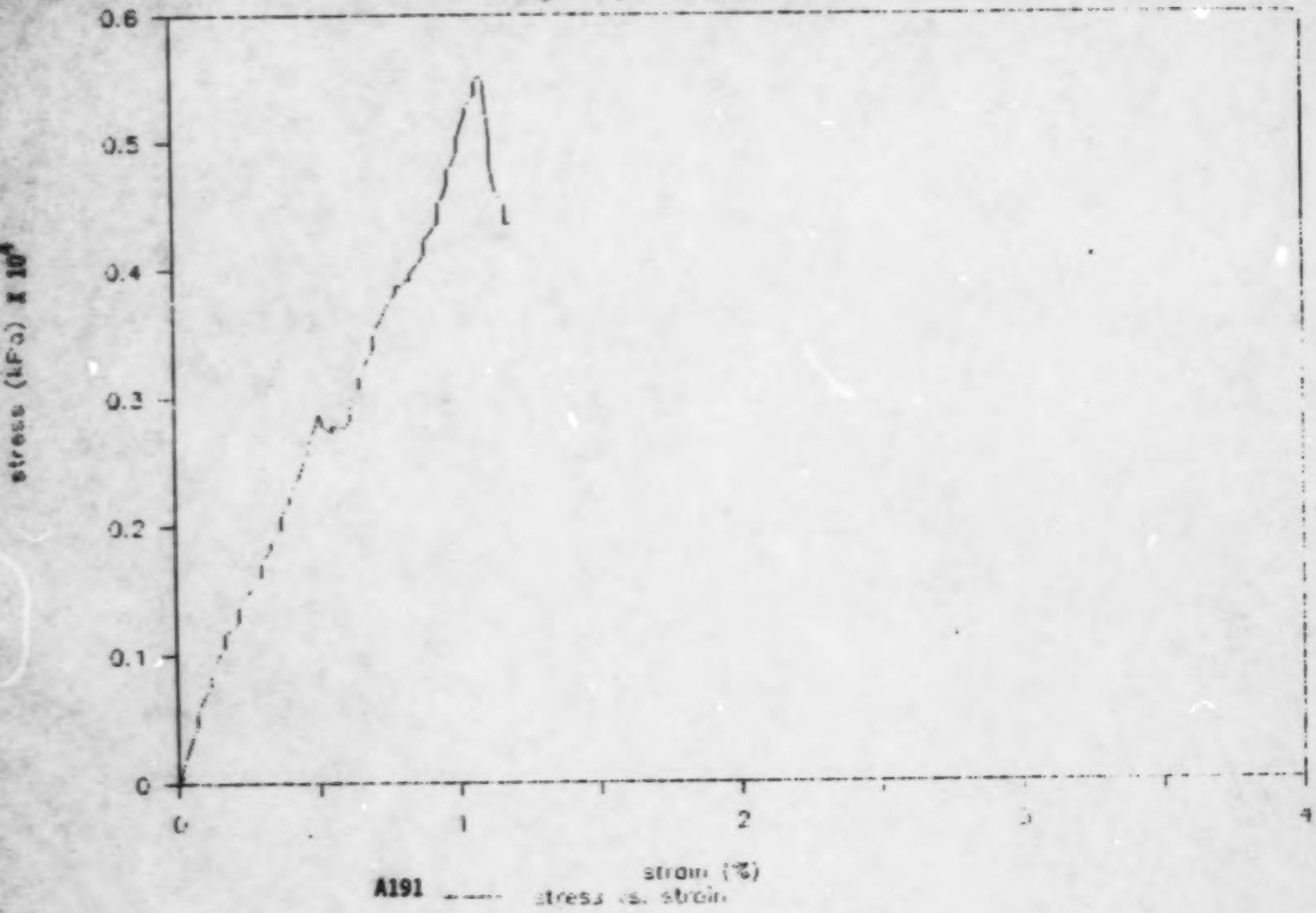
A189

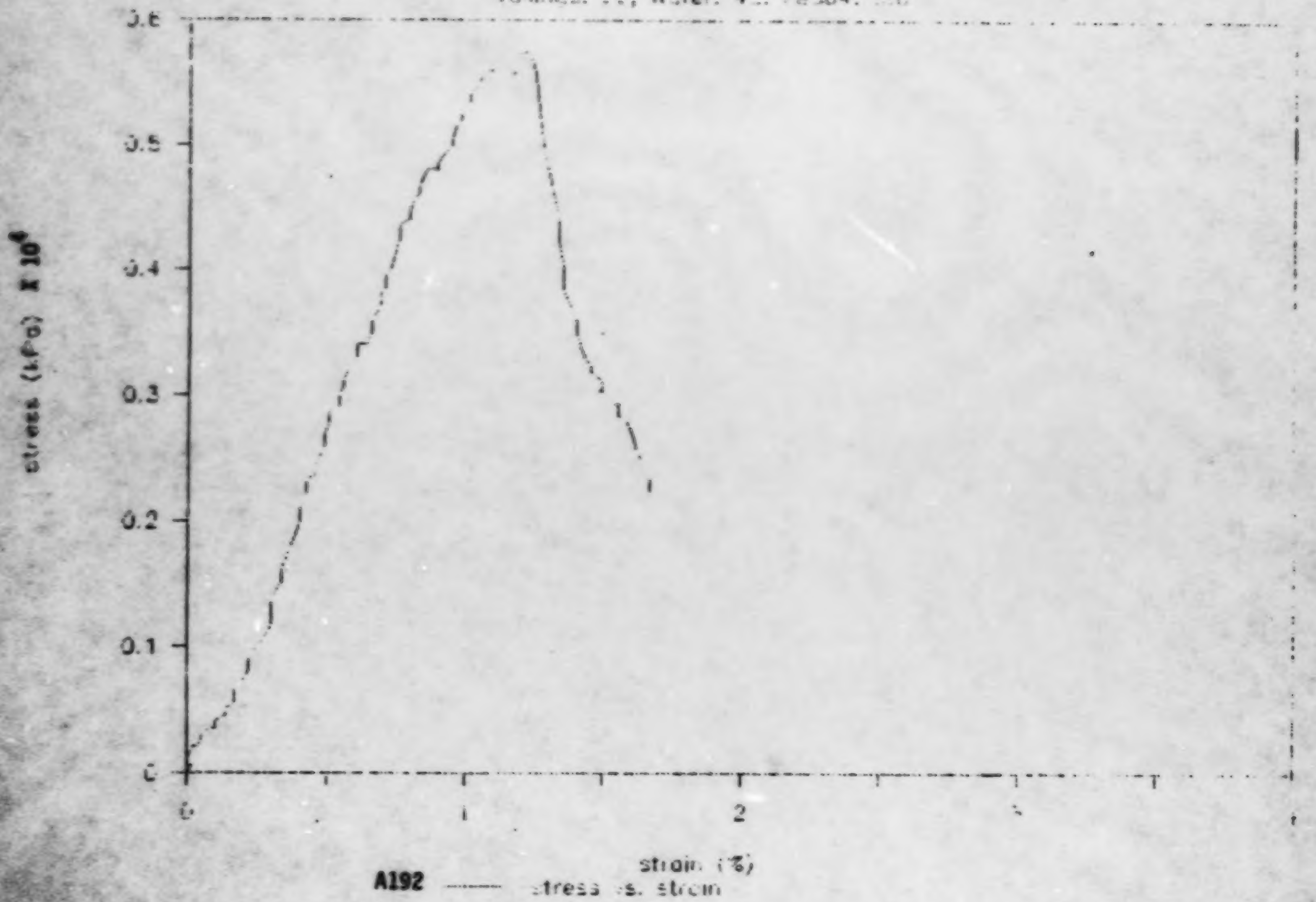


A190

−20 Mesh, 28 days; Anhydrite: 75 part

Tailings: 25; Water: 42; FeSO4: 3.6




stress (kPa) X 10⁶
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
strain (%)
A192
stress vs. strain